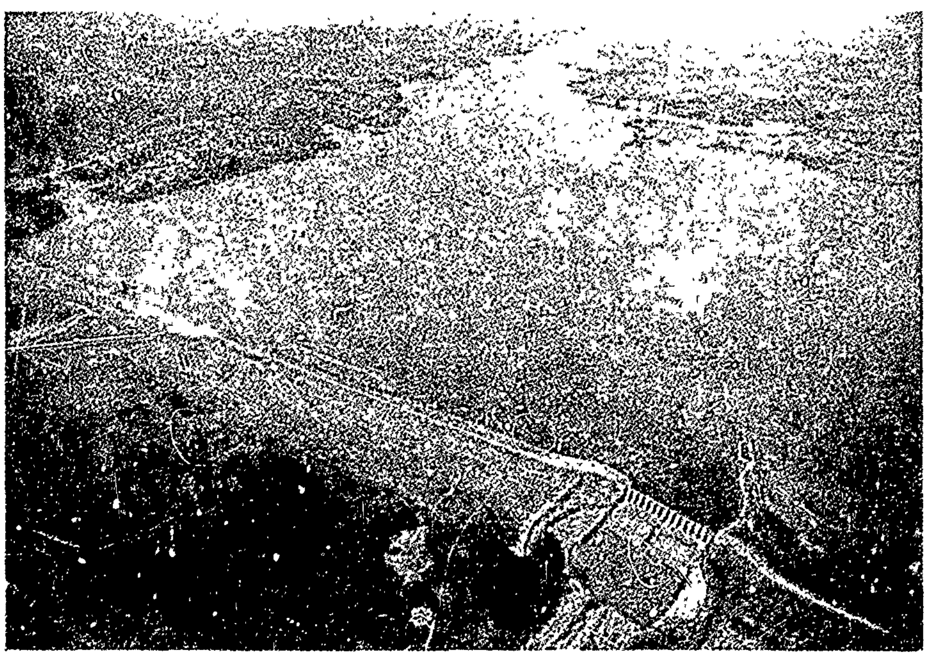
US Army Corps
of Engineers
Fort Worth District

Embankment Criteria, Performance, and Foundation Report

Waco Lake
Bosque River, Texas
Brazos River and Tributaries, Texas



January 1990

DEPARTMENT OF THE ARMY
FORT WORTH DISTRICT, CORPS OF ENGINEERS
P. O. BOX 17300
FORT WORTH, TEXAS 76102-0300

REPLY TO
ATTENTION OF

CESWF-ED-G

18 January 1990

MEMORANDUM FOR SEE DISTRIBUTION

SUBJECT: Waco Lake, Texas, Embankment Criteria and Performance Report

In accordance with paragraph 8 of ER 1110-2-1901, the subject report is transmitted for your use and information.

FOR THE COMMANDER:

Encl

for Ronald P Zunker
R. TERRY COOMES, P.E.
Chief, Engineering Division

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WACO DAM

BOSQUE RIVER, TEXAS

BRAZOS RIVER BASIN

EMBANKMENT CRITERIA, PERFORMANCE

AND

FOUNDATION REPORT

U.S. ARMY ENGINEER DISTRICT

CORPS OF ENGINEERS

FORT WORTH, TEXAS

JANUARY 1990

WACO DAM
BOSQUE RIVER, TEXAS

EMBANKMENT CRITERIA, PERFORMANCE,
AND
FOUNDATION REPORT

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WACO DAM

WACO, TEXAS

EMBANKMENT CRITERIA, PERFORMANCE,
AND
FOUNDATION REPORT

SECTION 1 - INTRODUCTION

1-01. **Purpose of Report**. This report is prepared with approval of Office, Chief of Engineers, in accordance with ER1110-2-1801, Construction Foundation Reports, and ER 1110-2-1901, Embankment Criteria and Performance Report, and is presented as a single report with every effort being made to satisfy requirements of both ER's. It was considered that combining information required in ER's 1110-2-1801 and 1110-2-1901 into a single report affords engineering and other disciplines a better understanding of design and construction problems related to the project. The report covers design, construction, and operational information on geotechnical features of the major elements of Waco Dam consisting of the embankment, spillway, and outlet works (See Photos 1 through 5).

The report summarizes available information on the investigation, design, and construction of the dam as well as operational performance of the project since its completion in 1965. Further, it presents information on the slide of a portion of the embankment during construction and information on redesign and reconstruction required for successful completion of the project. Significantly, the report evaluates

Keywords: Foundations structures; Structural geology/ground water; Embankments sliding/seepage/stability; Dam safety (EDC)

the performance of the embankment, particularly the critical phenomena such as foundation pore pressures. Detailed supporting documents are available for review in the Geotechnical Branch, Fort Worth District, and at the Waterways Experiment Station. Additional information is also available in papers listed in the Selected References.

1-02. <u>Construction Authority</u>. Congressional authority for the construction of Waco Reservoir, a unit in the plan of improvement for the Brazos River basin, is contained in the Flood Control Act approved 3 September 1954 (Public Law 780, 83rd Congress, 2nd Session) in accordance with the plan as outlined in House Document No. 535, 83rd Congress. Authority to initiate advanced planning was contained in the Public Appropriation Act of 1956, approved 15 July 1955 (Public Law 163, 84th Congress, 1st Session and in Advice of Allotment C-29, 3 August 1955).

1-03. <u>Location and General Description</u>. The Waco Dam is located on the Bosque River, in central McLennan County, immediately northwest of the city of Waco (See Plate 1). The new Waco Dam embankment and appurtenant structures replaced a previous dam constructed in the early 1930's, located approximately 3,000 feet upstream. The spillway gates and the bridge over the spillway were removed from the old dam. The remaining embankment of the old dam had a crest wall at elevation 432, which is below the conservation pool of the new lake. The watershed is approximately 89 miles long, 19 miles wide, and includes the drainage systems of both the North and South Bosque Rivers. These two rivers

converge about one mile upstream from the dam. The dam provides flood protection and water supply for the city of Waco and also furnish protection for approximately 1,188,000 acres of floodplain on the Brazos River downstream from Waco. For location and general plan of project, see Plates 1 thru 3. Pertinent data concerning the project are tabulated below:

DAM:

Type, rolled earthfill:

Length, overall . . . . 18,045 ft (including dike & spillway)

Height, above streambed . . . . . 140 ft

Width, crown . . . . 20 ft

SPILLWAY:

Type, ogee, gate controlled:

Length, at crest. . . . . 560 ft

Total length. . . . . 904 ft

Crest elevation, msl . . . . . . 465 ft

Gates, 14-tainter . . . . 40 ft x 35 ft

Top of gates, elevation msl . . . 500 ft

Outflow, capacity at maximum
design water surface. . . . 622,900 cfs

OUTLET WORKS:

Type, gate controlled conduit:

Gates, 3-tractor type . . . . 6 ft 8 in X 20 ft

Conduit, diameter, ... 20 ft

Invert elevation, msl . . . . 400 ft

Outflow, capacity at top of flood control pool . . . 20,100 cfs

1-04. Project History. Initial investigations for the project document site were made during 1940 in the general area of river mile 4.6 on the Bosque River. During this period 12 NX core holes, totaling 833.3 linear feet, and 22 auger holes, totaling 317.3 linear feet, were drilled for foundation study. The information developed from this study was used to secure authorization of the project as a unit in the plan of improvement for the Bosque River basin.

In 1955 studies were made on alternate damsites located both upstream and downstream from the project document site. These studies considered a dam at mile 7.35 on the North Bosque River in combination with a dam at mile 5.3 on the South Bosque River; a single dam at existing Lake Waco Dam; and a single dam at mile 1.5 on the Bosque River. Foundation conditions at each of these sites appeared to be suitable for the proposed structures. However, after economic studies were completed the project document site, with minor alignment changes, was selected for the Waco Reservoir project. A construction contract for the first increment of the embankment was awarded in June 1958. Deliberate impoundment began in February 1965 and the lake reached elevation 455 (top of conservation pool) in May 1965. The project was originally scheduled for completion in June 1964. However, a major slide occurred in the embankment in October 1961 which necessitated redesign of a portion of the embankment and also resulted in redesign of the spillway.

1-05. Foundation Slide. A major slide involving part of the embankment and its foundation occurred in October 1961, during construction of the first major portion of embankment under Contract No. 60-82 (See Plate 47 and Photos 6 and 7). The portion of embankment affected was between Stations 50 and 65. According to the classification system of the U.S. Committee on Large Dams, (ASCE/USCOLD, 1975), the slide was a "Type 3 Accident"; a slump that took place before any water was impounded. A detailed description of the slide is given in paragraph 5.02.

1-06. Contracts and Contract Supervision.

a. The Waco Dam was designed by and constructed under supervision of U.S. Army Engineer District, Fort Worth, Texas. Field supervision was under the Waco Area Office.

b. Geotechnical personnel were not assigned to the project until after the embankment slide occurred. After that event, all foundation preparation, mapping, treatment, and grouting, and also all instrumentation installation, monitoring, interpretation, and embankment zoning and control, were performed under supervision and final approval of geotechnical personnel assigned to the project office from the Fort Worth District Office.

c. A board of consultants was appointed to review all investigation, design, and construction activity after the embankment slide occurred. Board members were as follows:

Dr. Arthur Casagrande
Professor of Soil Mechanics
Harvard University
Cambridge, Mass.

Mr. Hibbert M. Hill
Chief Engineer
Northern States Power Co.
Minneapolis, Minn.

Mr. Edward B. Burwell, Jr.
Consulting Geologist
Upperville, Virginia

d. Listed below are the major contracts for the construction of the project structures:

Contract No.	Contract Title	Contractor	Awarded
58-530	Initial Embankment	Dean Skinner Austin, TX	13 Jun 58
60-82	Portion of Embankment and Partial Excavation of Spillway	Moorman and Singleton Wills Point, TX	2 Sep 59
60-255	Outlet Works	Hoffman and Borders Waco, TX	6 Nov 59
61-963	Spillway	Elmer C. Gardner Inc., Houston, TX	23 Mar 61
62-1257	Slope Protection	F. L. Vollintine Fort Worth, TX	3 Apr 62
62-1399	Repair of Embankment	R. G. LeTourneau, Inc. Longview, TX	7 Jun 62
63-1342	Clearing	E. D. Robinson Cascade, CO	12 May 63
63-1462	Completion of Embankment and Construction of Service Bridge	Clement Bros., Co. Hickory, NC	28 Jun 63
64-174	Maintenance Facilities and Access Road	Commercial Constr., Co., Waco, TX	24 Sep 63

SECTION 2 - FOUNDATION INVESTIGATIONS

2-01. **Investigations Preceding Construction**. Foundation investigations for design memoranda and final design were completed during the period from 1955 through 1961. These studies consisted of drilling auger and core borings along the proposed center line of the embankment and at selected locations beneath the proposed spillway and outlet works. In addition to the foundations borings, auger holes were drilled in proposed borrow area locations.

2-02. **Investigation During Construction**. A large number (734) of core, auger, fishtail, instrumentation, and inspection holes were drilled during construction as a part of the slide investigation and redesign of the embankment. Locations for most of the above referenced borings are shown on plates 11 through 14.

SECTION 3 - GEOLOGY

3-01. Regional Geology.

a. Physiography. The project is located on the western margin of the Balcones fault zone. This northeast-southwest trending zone of normal, locally en echelon faults marks the easternmost boundary of the Great Plains physiographic province (Grand Prairie sub-province) and the westernmost boundary of the Gulf Coastal Plain physiographic province. The Brazos River comprises the prime drainage system in the region with the Bosque, Leon and Little Rivers being major tributaries.

b. Stratigraphy. Ranging from west to east, geologic formations cropping out include limestones, shales, and sandstones of the Trinity, Fredericksburg, and Washita groups of the lower Cretaceous and the upper Cretaceous, Gulf series. Regional geology is shown on Plate 24.

c. Structure. The regional structure is that of a homocline broken by the faulting of the Balcones fault zone. The regional dip of the Cretaceous rocks west of the fault zone is 20 to 30 feet per mile to the southeast. At the Balcones fault zone, the regional dip of the formations increases to 60 to 90 feet per mile to the southeast.

d. Ground Water.

(1) General. Regional ground water is of two types: free or unconfined ground water that accumulated primarily in alluvial deposits, and confined or artesian ground water which

is ground water having sufficient head to rise above the aquifer in which it is encountered but which does not necessarily rise above the ground surface in a well.

(2) **Free Water.** Free water is found in the dam and reservoir areas near the ground surface in valley alluvial deposits and frequently in the basal beds of river terrace deposits that overlie impermeable bedrock strata. Free water recharge results from percolation of rainfall and migration of ground water through the alluvium. Domestic surface wells producing from this source in the area often go dry in drought periods.

(3) **Artesian Water.** There are five sources of artesian water present in the bedrock strata of the Waco region. They are, in descending order: The Edwards Limestone, the Paluxy Sand, the Glen Rose Limestone, and Hensel and Hosston Sands of the Trinity Group. The Edwards Limestone is the only aquifer outcropping in the reservoir area. Each of these aquifers is separated from the other by relatively impervious formations. Piezometric elevations (developed from water well inventory data) range from +350 to +400 MSL. The piezometric contours developed from water levels of the Glen Rose, Hensel and Hosston Formations show a definite cone of depression in the Waco municipal and industrial area with the dam site located on the northwest flank of this depression. Based on this information, it is concluded that the high readings obtained from piezometers installed at the

dam site were not the result of artesian conditions in these formations. Further, a test hole drilled 300 feet downstream from the dam on Station 56+00 penetrated all formations from the top of the Georgetown to the top of the Glen Rose. The piezometric surface in this open hole was established at elevation 418 or approximately one (1) foot above ground surface. Therefore, this section of formations is not considered to be contributing to the high pressures recorded from shales in the piezometers at the dam site.

3-02. <u>Site Geology.</u>

a. <u>Physiography.</u> At Waco Dam the Bosque River impinges against the Bosque escarpment, a prominent cuesta of Austin Chalk that forms the south wall of the river valley. The present valley of the Bosque River has a broad floodplain about 6,500 feet wide, between a low, gentle bluff to the north and a cuesta to the south. A geologic and topographic map of the dam and reservoir is shown on Plate 25.

b. <u>Stratigraphy.</u>

(1) <u>General.</u> Bedrock formations which constitute the foundation of the dam and its appurtenant structures are, in ascending order, the Georgetown Limestone, Del Rio Shale, Pepper Shale, Eagle Ford Shale and Austin Chalk, all of Cretaceous Age. A geologic map of the dam and reservoir is shown on Plates 11 and 25, and geologic sections are shown on Plates 26 through 34.

(2) <u>Georgetown Limestone.</u> The only Georgetown

Limestone exposed at the dam was the upper portion of the formation encountered in excavations for the spillway stilling basin. The nearest outcrop is in the South Bosque River valley, 4 to 5 miles northwest of the dam. A deep boring located 300 feet right of the dam axis, Station 56+00, indicates the formation is about 158 feet thick. The Georgetown penetrated in this boring was the most competent material encountered at the project and consists chiefly of argillaceous limestone. The upper portion of the Georgetown includes many alternating beds of argillaceous limestone and calcareous shale. The contact between the Georgetown and the overlying Del Rio Formation is gradational, but has been correlated between borings using electrical resistivity logs and core. There is no break in deposition apparent between the Georgetown and Del Rio Formations.

(3) <u>Del Rio Shale.</u> The Del Rio Shale is exposed in the spillway and along the spillway discharge channel, and is present beneath the overburden throughout much of the left abutment from the spillway to the north fault. Also, it is the uppermost bedrock between the north and the south faults beyond 2,500 feet right of the dam axis. The Del Rio is a remarkably homogeneous and distinctly calcareous clay shale, having three easily identified marker horizons: A thin (0.05-0.20-foot thick), tan-white bentonite seam located approximately 3 feet from the top, and two marly (very calcareous) horizons, approximately 18 and

22 feet above the base. Occasional, thin (0.1 to 0.5 foot thick), lenticular limestone beds are scattered throughout the formation. The basal contact of the Del Rio Formation has been placed 0.5 foot below a marly limestone bed about 0.8 foot thick at the bottom of the formation. The top of the Del Rio is an erosional surface. The erosional interval was of unknown duration, but erosion was sufficient to remove most of the Buda Limestone Formation that originally overlaid the Del Rio. Residual detritus, consisting of fossil shells, sand, shale pebbles, and phosphatic nodules, cemented into a limestone conglomerate, overlies much of the erosional unconformity at the top of the Del Rio. The secondary mineral marcasite is present throughout the Del Rio in nodular form and as fossil replacements. The thickness of the Del Rio varies from 64 to 72 feet. Photo 8 shows the contact of the Del Rio Shale and the Pepper Shale above it.

(4) **Pepper Shale.** The Pepper Shale is the uppermost primary formation between the north and south faults both immediately upstream and downstream from the dam axis. Within the general area of the dam, the total thickness of the Pepper Formation ranges from 60 to 80 feet. Locally beneath the dam the Pepper is as thin as 40 feet where it has been eroded and covered with overburden. Lithologically, the Pepper is a dark gray to black, fat, waxy, fissile, possibly carbonaceous, compaction clay shale, some zones of which are soft to very soft. These

uncorrelatable soft zones can ordinarily be attributed to structural rather than lithologic causes. Pepper shale rapidly deteriorates on wetting and drying. With the exception of some slightly calcareous interbeds, the Pepper Shale is noncalcareous. However, calcite is occasionally found as a precipitate along slickensides in the shale and within the fault planes. The Pepper is a bentonitic clay shale, but no relatively pure bentonite in the form of seams or bands is present in the unit.

Secondary ferric staining in the form of jarosite, hematite, and limonite is prominent on exposed, weathered Pepper outcrops. The mineral marcasite is common throughout the Pepper, both finely disseminated and associated with the secondary lithologic characteristics. The Pepper Shale contains thin seams of siltstone, sandy siltstone, and sandstone. Clay-ironstone occurs only as concretions. Generally, these seams are light gray where encountered, occasionally grading to gray-brown or brown in color. They oxidize to yellow-tan or reddish-brown when exposed at an outcrop. No persistent seams were noted in the upper 8 feet nor the lower 6 feet of the formation. Materials described in core logs as hard, tan siltstone, usually 0.1 to 0.2 foot thick, are better classified as clay-ironstone based on outcrops and on other descriptions of the Pepper Shale. This material occurs as disk-shaped concretions in a number of horizons within the Pepper Shale and its lateral equivalent, the Woodbine Formation. Clay-ironstone is comprised largely of clay cemented

with the mineral siderite (iron carbonate) and is reported to contain some disseminated, microcrystalline pyrite. The Pepper is unconformably overlain by the Upper Cretaceous Eagle Ford Group. A typical section of the Pepper-Eagle Ford contact is shown on Photo 8.

(5) **Eagle Ford Shale.** The Eagle Ford Shale is a stratigraphic group consisting of two formations: The Lake Waco (lower) and the South Bosque (upper). Exposures of the Eagle Ford are shown on Photos 10 and 11.

(a) **Lake Waco Formation.** The Lake Waco Formation occurs immediately beneath the overburden from the river fault (centerline Station 9+25) to the south fault, and between the south and north faults, upstream from the dam axis. It is characterized as a high lime content, fossiliferous shale with many marl, argillaceous limestone and bentonite seams. Most of the shale contains disseminated fossil fragments and microfossils, giving the dark beds a "salt and pepper" appearance. Plant imprints and thin lignitic remains are prevalent.

At least 80 bentonite seams have been noted in the formation, ranging from a trace to a foot or more in thickness. They vary from light gray to dark gray; include an abundance of coarse-grained material; and occasionally are micaceous or sandy. Many of the bentonites are persistent, and are correlatable throughout the dam site area.

The base of the Lake Waco Formation is marked by a thin, somewhat lensy detrital conglomerate of pebbles, shell and fish fragments, sand, shale fragments, and limestone nodules in a lime matrix. Occasional joints and fractures with some calcite fillings are encountered. Marcasite is found throughout the formation, particularly within lime concentrations and bentonites.

(b) South Bosque Formation. The top o' 'he uppermost marly zone encountered in borings was chosen to be the contact between the Lake Waco and the South Bosque for stratigraphic interpretations at Waco Dam. Using this contact, the South Bosque averages 100 feet in thickness at the dam site. It is immediately beneath the overburden in the Bosque valley from the river fault (approximate Station 9+25 on centerline) to the right abutment. Lithologically, the South Bosque consists of a compaction shale of low lime content and very few lime concentrations. Occasional sandy seams and horizons are present, but generally they cannot be correlated over any appreciable area. The shale is dark blue-gray to black, noncalcareous to slightly calcareous, soft, fissile, thin bedded, locally silty or sandy, and slakes slowly on exposure. It contains occasional invertebrate fossils and fish fragments and a few thin concentrations of shellfish. No bentonite beds were found in the formation. The top of the South Bosque Formation is an erosional surface, which is overlain by a thin, discontinuous conglomerate.

The base is marked by a sandy horizon 1.5 to 3 feet thick that overlies the first sandy marl of the Lake Waco.

(6) Austin Chalk. The Austin Chalk is exposed in the right abutment of the dam site. In outcrops, extensive weathering of the Austin is evident to a depth in excess of 15 feet. The Austin is considerably more resistant to erosion than are the underlying shales; consequently, it forms a prominent escarpment. The formation contains many argillaceous zones up to 8 or more feet thick that are less resistant to erosion than is the chalk itself. At the dam site, two bentonite seams separated by a thin (0.1 foot) chalk bed are present approximately 44 feet above the Austin/Eagle Ford contact. The upper bentonite is 0.70 to 0.90 foot thick, and the lower one is 0.10 to 0.15 foot thick. Both are light gray and are persistent throughout the area. The Austin also contains nodular and/or disseminated marcasite, as well as calcite coatings on slickensides and fractures. In general, the Austin is comprised of limestone which is soft to moderately hard, both thin and massively bedded, argillaceous, jointed, chalky, and is medium to light gray in color. Total thickness of the Austin at the dam site is about 128 feet. The Austin Chalk, as it was exposed in the excavation in the right abutment, is shown on Photo 26.

(7) Overburden. The overburden at the dam site consists of Brazos River terrace deposits of Pleistocene age and recent Bosque River valley alluvium. Terrace deposits occur on

the left abutment and extend beyond the end of the dam. Composition of the Brazos terrace deposits differs from that of the alluvial deposits of the Bosque River. The basic difference between the two deposits is the high percentage of siliceous material in the Brazos gravel as compared to predominantly carbonate materials (limestone pebbles and pelecypod shells) of the Bosque River gravel. Bosque River gravel is of commercial quality and has been extracted extensively in an area between Stations 24+00 and 75+00 downstream from old Lake Waco dam to the Bosque River where it crosses to the north side of its valley. Abandoned gravel pits occupied at least 75 percent of this area prior to construction. Conglomerate beds are widespread in the lower portion of the gravel deposits as extensive, lens-like masses, usually being less than 4 feet thick.

Blasting was necessary to remove conglomerate in the spillway approach channel and in the excavation for the outlet works. Conglomerate in the embankment foundation downstream from the centerline adjacent to the outlet works was not removed as it was undesirable to use explosives close to the outlet works structure. Stripping of the overlying alluvial material in the pits generally ranged from 3 to 10 feet and, after removal of the gravel, the stripped materials were used to backfill the pits.

Alluvium of which the floodplain is comprised consists of clay, sand, and some fat clay varying from 3 to 35 feet in thickness.

3-03. Weathering. Weathering of the foundation rocks at the dam site was of little engineering significance except in the Austin Chalk which forms the right (south) abutment. For the most part, the depth of weathering in the Georgetown, Del Rio, Pepper, and Eagle Ford shales and limestones extends only a few inches below the surface of the bedrock. In outcrops these formations weathered to a depth of from 1 to 5 feet except where much jointing or fracturing is present. In these areas weathering extends as much as 6 to 10 feet below the bedrock surface, primarily as stained halos along the joints and fractures, but the depth of weathering was at least 20 feet at the top of the right abutment and as much as 30 feet in the abutment slope.

Where exposed in required excavations, Cretaceous shale formations were protected from drying and slaking. Protection was provided by covering the shale with gunite or asphaltic membranes immediately following excavation.

3-04. Structural Geology.

a. Faults. Structural geology at Waco Dam directly involves the Balcones fault zone, the western boundary of which is believed to be the north fault at the dam. All faults of the Balcones system are of normal (gravity) type, meaning that they produce separation and down-dropping of bedrock blocks rather than overlapping and crowding of the blocks involved. Regional dip at the dam site is about 50 feet per mile toward the

southeast. Locally, bedrock dip is frequently inconsistent with regional dip because the tilt of individual fault blocks. Faulting at the dam site consists of three major faults. These faults are all downthrown to the southeast. They are identified as the north, south, and river faults. Some subsidiary synthetic and antithetic tearing is associated with them. They are shown on Plate 50. The north and south faults were extensively investigated by trenching as shown on Plates 51, 52 and 53. Drag along the faults was found to be well developed. Fault drag is the locally steep dip of the bedrock marginal to a fault resulting from blocks of bedrock sliding past each other along the fault. In some cases, fault drag has distorted the shales so severely that bedding was completely obliterated. This was found most frequently in the Pepper Shale. In cases where both the hanging wall and footwall materials were soft (i.e., Pepper faulted against Del Rio), the fault plane was poorly defined because of crushing and intermixing of material from both formations. Fault planes in the Pepper and Del Rio change dip in some locations, usually due to differences in strength of the beds through which faults pass. The softer and weaker shale beds were frequently found to be crumpled into a soft, clayey breccia. The more brittle Lake Waco portion of the Eagle Ford retains its bedding characteristics better than the other shales and forms a more distinct fault plane with less brecciation. The relatively strong Georgetown Limestone showed little drag along

faults and had more consistent dips of the fault planes penetrating it. Regional studies of the faulting indicate that the faults penetrate the entire Cretaceous section of the area.

(1) North Fault. The north fault crosses the axis of the dam at Station 58+40, (See Plates 50, 51 and 53). Displacement of this fault varies from 120 feet in the vicinity of 3,000 feet right of centerline to 92 feet in the vicinity of 2,000 feet left of centerline. The north fault is poorly defined beneath the overburden, probably because of softening of the Pepper and Del Rio shales by ground water. The dip of the fault, estimated to be 73 degrees, is probably this steep because of resistance to shear of the Georgetown LImestone, which is very near the surface of the footwall of the fault. Tearing along the north fault is not prevalent. Two synthetic tears were noted downstream in the drainage trench excavation, very close to the north fault. Generally, the soft shales at the contact between the bedrock and the overburden at the north fault lend themselves more readily to warping and "remolding" along the fault plane, so tearing is held to a minimum. Small blocks of limestone are occasionally found along the fault plane and are generally considered to be fragments of the limestone lenses found in the Del Rio or fragments of the detrital conglomerate at the top of the Del Rio. Crystals and crystalline sheets of secondary calcite are found precipitated between slickensided surfaces of the fault plane. Oil stains are often found associated with the

slickensides. Weathering and oxidation are minor along the fault plane, with infrequent oxidation stains encountered at depth. A geologic map of trenches which exposed this fault is shown on Plates 51 and 53.

(2) South Fault. The south fault crosses the axis of the dam at Station 51+50 (see Plates 50, 52 and 53). The fault is also a normal or gravity fault. Pepper Shale forms the footwall of the south fault. Where observed upstream from the dam axis, a thin veneer of basal Eagle Ford overlies the Pepper between the north and south faults. Geologic maps of trenches in which the south fault is exposed are shown on Plates 52 and 53. Originally, it was believed that the north and south faults converged near the embankment of the old Lake Waco Dam. However, later investigations showed that the south fault divides into two faults beneath the upstream portion of the dam, one of which continues upstream, apparently striking parallel to the north fault. However, insufficient data are available to trace this south fault member beyond 800 feet left of centerline. The other member of the divided south fault converges with the north fault and will be referred to later as a convergent tear fault. Average displacement on the South Fault is 92 feet, with variations from 68 feet at 800 feet left of centerline, to 111 feet at 3,000 feet right of centerline. Displacement of the member fault converging with the north fault diminishes as it approaches the north fault. See Plates 26 and 27 for sectional

views of these faults, and Plates 11, 12, and 50 for plan views of the faulting. Brecciation along the south fault is mostly associated with the Pepper Shale in the footwall of the fault. Drag dips of 30 degrees or more are common along the hanging wall of the south fault. Fault drag phenomena were found 200 feet or more down dip from the fault trace in the hanging wall of the fault. The most pronounced warping is found in the first 85 feet down dip from the fault trace. Between 85 and 200 feet distant from the fault trace, dip anomalies of less than 5 degrees occur. The estimated dip the south fault at the overburden contact is 48 degrees. The dip increases with increasing depth as the deeper Georgetown Limestone is approached. Locally, as many as five tears have been found in a distance of 30 feet. These tears are not always shown on foundation maps because they often parallel the bedding in the drag zone near the fault and, although observable in excavation walls, are extremely hard to recognize at the surface. The only antithetic tears found at the site are associated with the south fault. Indications are that the drag-warped Eagle Ford in the hanging wall of the fault has a markedly greater permeability from shattering of its harder beds. Downstream from the embankment some of the large diameter sand drains drilled into the Eagle Ford made considerable water. Occasional streams up to an inch in diameter were noted to emerge from open fractures in the wall of the borings. This open condition in the Eagle Ford apparently was absent in the area

upstream from the embankment as test grouting encountered relatively tight conditions. Some petroleum residue and the mineral calcite were found along the bedding planes and slickensides in the faulted area. Weathering is not prominent along the fault planes, but occasional iron staining is found in the breccia, and the bentonites show considerable oxidation for a few feet downdip from their contact with the overburden.

(3) River Fault. The primary plane of the river fault is located at Station 9+10 on the embankment centerline, but the fault has disturbed an area of the shale foundation between Station 9+10 and Station 11+50. Shallower than the Georgetown Limestone, the river fault is a complex of faults rather than a single fault. Faults comprising the complex are numerous in the Pepper and Eagle Ford Shales, and diminish in number downward toward the Georgetown LImestone. Electrical logs from borings of three sections crossing the river fault complex reasonably established that a single fault within the Georgetown Limestone split progressively into many faults in the shales overlying the Georgetown. Lithologic sections EE (Plate 28), PP (Plate 34), and FF (Plate 29) are drawn symbolically in part because of the complexity of the faulting. In tracing individual faults up dip to the northwest from the Georgetown, the dip of faults was steep in the Georgetown diminishing upward to at least 45 degrees in the shales overlying the Georgetown, and steepening somewhat again in the Lake Waco portion of the Eagle Ford Group.

Differences of fault dip seem to reflect differences in shearing resistance of the materials through which the faults pass. The river fault has a dip of at least 65 degrees, with dips of as little as 30 degrees in the shales above the Georgetown Limestone. At the top of the bedrock, the Eagle Ford occurs in both the hanging wall and the foot wall of the fault. About halfway across the fault complex, the Eagle Ford is faulted downward sufficiently that the South Bosque is immediately beneath the overburden. North of this location the Lake Waco is beneath the overburden (see Plate 11). The Lake Waco/South Bosque contact has not been shown on the geologic sections of this report. The river fault has not been traced beyond the limits of the embankment foundation with any degree of accuracy, although it has been projected 2,900 feet downstream on the basis of a line of holes drilled for a road relocation. Average displacement of the river fault is about 73 feet, but locally is as great as 80 feet. No excavations have exposed this fault and its bounding materials for examination. Caliche deposited in fracture planes in microcrystalline sheets was seen in cores. Weathering is not significant along the fracture planes or in the brecciated material.

b. <u>Other Structural Conditions.</u> Stripping for the embankment foundation uncovered a brecciated area between Stations 31+00 and 34+00 (see Photo 10, Exhibit 6). Its relationship with other geologic structures in the area is

uncertain. The brecciated materials were soft, moderately weathered, and included a moderate amount of free water. The brecciated zone varied from 1 to 12 feet in width, striking approximately N 69 degrees W with an apparent dip of 50 degrees in a reverse (NE) direction to what would be considered normal for the area. Electric logs from borings on either side showed no displacement, and there was no evidence of strike-slip movement.

Slickensides are common throughout all the shales at the dam site. Although the "slicks" are associated with faulting, it is believed that much of the phenomena can also be attributed to minor differential displacements occurring during consolidation of the materials. There is an abnormally greater number of slickensides in the Pepper Shale in the block between the north and south faults. This may be an effect of the short distance between the two faults, or it might be related to the difference in dip between the north and south faults. This difference may have produced some stress relief during the early history of the faults. See Sections E-E, F-F, and G-G, Plates 28, 29, and 30.

Jointing was noted at the dam site on exposed bedrock surfaces and is most prevalent in the Eagle Ford. Three joint systems are present. Orientation of these is essentially the same in all three foundation shales. Their strikes, plus or minus 5 degrees, average N79 degrees E (primary), N3 degrees E (secondary), and N43 degrees W (tertiary). Some joints in the

Eagle Ford have allowed formation of "rathole" solution channels. None of these have been traced for more than a few feet along the joints. The channels apparently enter the formation on joints within the weathered zone and develop laterally. Jointing had a minor effect on construction. Some overexcavation was required in the outlet works foundation to remove weathered material along joints. In the spillway excavation, relief of lateral restraining pressures allowed joints in the Del Rio to open; consequently, a nominal amount of sloughing or spalling took place. While drilling pressure relief wells under the spillway structure, some joints were encountered that allowed communication of drilling fluids to the surface. Slumping of the Austin Chalk and jointing within the rock had considerable effect on excavation in the right abutment.

Abnormal bedding dips at the dam site are attributed to faulting. Drag-increased dip marginal to fault planes has been discussed previously. The dip or tilt of individual faulted blocks of bedrock differing from regional dip is, almost universally, the result of variation of displacement along the faults bounding the individual blocks.

3-05. <u>Ground-Water Conditions.</u>

a. <u>Free Water.</u> Prior to construction, the free water in the alluvium was recharged by precipitation, and leakage from the old Lake Waco. Precipitation on the left abutment percolates through terrace deposits to bedrock then flows riverward. Little

rainfall accumulates in the right abutment as little alluvium is present within which it might collect and because the bedrock beneath is Austin of which only part is fractured. Storage capability of the fractured portions of the Austin Limestone is minor as fractures in the rock comprise a very small percentage of the rock mass. Fracturing has enhanced drainage of the limestone, however. Runoff is dominant over retention of precipitation on this abutment. The area contributing either ground water or runoff to the Bosque River is greater on the left abutment than on the right abutment. Ground water flowed downstream freely in the alluvial sand and gravel in the floodplain prior to construction. This flow was completely changed by construction of the dam. A cutoff trench was constructed beneath the dam from the right abutment to Station 3+50 and from the river channel (Station 15+00) to the spillway structure. The embankment foundation between Station 3+50 and the river channel in which no cutoff was constructed, is comprised of 30+ feet of impervious clay overburden which did not require cutting off. No cutoff was constructed beneath the dike that extends landward (up station) from the spillway on the left abutment.

SECTION 4 - STRUCTURES

4-01. Outlet Works Foundation.

a. Introduction and Description of the Structure. The outlet works is located approximately 1,050 feet northwest from the river channel and intersects the embankment at Station 24+00. The structure consists of a gated intake tower rising 145 feet above the invert, a 20-foot diameter conduit 497 feet long and a conventional stilling basin with a parabolically curved concrete apron (See Plates 6, 7 and 8). Two rows of baffles and an end sill dissipate the discharge energy. Water supply to the city of Waco is furnished through two 54-inch diameter conduits, one on either side of the primary conduit (See Plate 7). The construction contract (DA 41-443-CIVENG-60-255), was awarded to Hoffman & Borders, Inc., of Waco, Texas, on 6 November 1959. Work commenced 1 December 1959 and was completed 29 September 1962. The contract was bid at $1,825,743.00, and a final settlement of $1,943,540.76 was made on 1 November 1962.

During the period in which the outlet works foundation was exposed, the District Office did not have qualified personnel on site for foundation mapping. However, foundation data was obtained from construction records, personal observations, and communication with the construction representatives. Information thus obtained is used as a record of the as-built foundation conditions.

b. Foundation Excavation. Preparation of the foundations was performed in two operations: overburden stripping and bedrock excava-

tion. The overburden material consisted of about 3 to 10 feet of sandy clay and topsoil overlying 3 to 9 feet of sand and gravel. The sands and gravels included lenses of well cemented conglomerate up to 3 feet in thickness, generally lying in contact with the top of the bedrock. The spoil from the existing gravel pit strippings, consisted of an unstratified, heterogeneous mixture of alluvial materials that had been dumped behind the working face of the pits. This backfill was not usable as impervious material because it included blocks and fragments of conglomerate and excessive organic material. None of the structure was founded on or placed against overburden material.

Except for the cemented conglomerates, the overburden was removed by conventional methods using dozers and scrapers. The conglomerate generally required blasting for removal. All suitable clay materials were used in the temporary cofferdam built around the structure. The sands, gravels, conglomerate fragments, and other unsuitable materials were disposed of in a designated waste area located approximately 800 feet upstream from the intake structure.

Bedrock excavations varied in depth from 5 to 25 feet, depending on the shape of the base of the structure. The foundation for the approach walls, slab, and intake structure footings were founded about 5 feet below the top of bedrock, resulting in a required cut of about 9 feet at the discharge end of the conduit. The deepest bedrock excavations were in the stilling basin, which was carried approximately 25 feet into bedrock. Excavation methods were varied to suit conditions.

All vertical faces except oblique corners were precut with a coal saw. The oblique corners were line-drilled prior to excavation. Most of the bedrock was ripped loose with a dozer and loaded out with a front-end loader. Portions of the intake structure and stilling basin foundation were broken with explosives to facilitate loading. All loose or questionable material was removed by hand methods prior to placement of protective materials.

c. Character of the Foundation.

The bedrock under the entire outlet works structure is lower Eagle Ford (Lake Waco) Shale. Weathering was shallow, seldom exceeding one foot deep. Occasional joints were weathered to a depth of 3 feet, but did not cause construction problems. Several thin bentonite seams were encountered in the stilling basin excavation. Care was taken to insure that the bentonites were not exposed in the floor of the excavation. This required a minor amount of over excavation, usually to remove blocks of shale that had broken loose above the seams. The bedding planes in the foundation dipped slightly downstream and riverward. This caused a minor problem of over excavation in the horizontal portion of the foundation as the blocky calcareous shale beds tended to break out on bedding planes, resulting in a series of step-like ridges. The only structural problem encountered was a minor, broken or brecciated area exposed for the footing at the end of the right wing wall of the stilling basin. This area was discovered during excavation of the footing and required about 4 cubic yards of over excavation. The zone was apparently discontinuous, as it was not encountered in other excavations.

Dewatering posed no problems. The ground water entered the excavation through seeps along exposed joints and gravel layers but was easily handled by conventional pumping (3-inch centrifugal pump), and a temporary cofferdam was built around the entire outlet works structure to hold out possible flood waters from the Bosque River. A flood gate was placed in the cofferdam to allow flooding of the excavation in the event overtopping was imminent. However, flood conditions did not develop during excavations, and the gate was not needed.

d. Foundation Treatment and Preparation. Foundation remedial treatment was unnecessary except for foundation for the right wingwall of the stilling basin. In this area, a zone of brecciated shale was considered unsuitable as a foundation material because of its low bearing strength. To correct this condition, approximately 4 cubic yards of material were removed and the excavation backfilled with mass concrete. A few short bars of reinforcing steel were inserted vertically into the mass and allowed to protrude 6 inches into the normal footing. A small amount of over excavation was performed at other areas prior to placement of protective slabs or structural segments. However, this treatment was insignificant.

Subsurface treatment consisted of anchor bars and drain holes in the stilling basin and wingwalls (See Plate 8). Anchor bars were made up from No. 11 reinforcing steel, 10 to 18 feet long, grouted into 4-inch diameter drill holes. The anchor bars that were tied into the concrete apron and stilling basin floor were grouted 10 feet into

shale on approximate 10-foot centers. The bars along the stilling basin wingwalls were grouted 18 feet into shale on 9-foot centers and angled into the foundation at a 1V on 2H slope from about elevation 367 (outside face of the wall). Drain holes for the concrete apron and stilling basin floor consist of 4-inch diameter holes drilled 10 feet into shale, slanted upstream at a 2V on 1H slope, and spaced on approximate 20-foot centers. The drain holes were not gravel packed or lined.

Surface treatment of final grade for the shale foundations consisted of 6-inch protective concrete slabs on all floors and slopes and pneumatically placed asphalt on all vertical faces. All final grades were covered as soon as possible after approval, and none were exposed longer than 4 hours. Deterioration of the prepared Eagle Ford surfaces was minor. No foundation grouting was required under the outlet works structure.

e. Investigations During and After Construction. A series of survey points were set in the invert of the conduit and were read periodically to check for possible elongation of the conduit during construction of and loading by the embankment. However, no significant anomalies were noted in the readings during this period.

f. Subsequent Work Related to the Outlet Works. Under the provisions of Contract No. DA-41-443-CIVENG-63-1462, (Completion of Embankment and Construction of Service Bridge), the approach and discharge channels were excavated and riprap was placed at the intake

and around the stilling basin. Both channels were excavated with dozers, rippers, and scrapers. No anomalies were noted.

The approach channel was excavated to elevation 400, with shale exposed in the channel for a distance of 425 feet upstream from the intake structure. Overburden in the approach channel was mostly random, uncompacted fill, reworked during gravel mining operations. Side slopes were constructed 1V on 3H through the upstream berm, leaving a 20-foot bench at the top of natural ground. Slopes of 1V on 2H were cut in the overburden.

The discharge channel was excavated with dozers, rippers, and scrapers. The first 100 feet downstream from the end sill was over-excavated 4.5 feet to accomodate 18 inches of bedding and 36 inches of riprap. The channel, starts at elevation 362 (top of riprap) at the end sill, rises to elevation 368 on a 1V on 10H slope, and remains at elevation 368 to the point where it discharges into the Bosque River. The 1V on 2H cut slopes have eroded excessively since closure and diversion of the river. The shale in the lower part of the channel has not been appreciably eroded or disturbed by discharge through the outlet works.

4-02. Spillway Foundation.

a. Introduction and Description of the Structure. The spillway is located on the left abutment of the dam approximately 7,200

feet northwest of the river channel (See Plates 9 and 10). The centerline of the spillway is at embankment Station 86+07. The spillway is comprised of an approach channel; a concrete gravity-type overflow weir with fourteen 35 x 40-foot tainter gates and 13 piers; concrete gravity-type nonoverflow sections which are enveloped by the earth embankment (See Plate 10); a control room located in the left abutment; a conventional stilling basin with a reinforced concrete anchored floor with baffles and end sill; and reinforced concrete cantilever training walls. The preliminary excavations for the intake, discharge channels, and the structure foundation were performed under Contract No. DA-41-443-CIVENG-60-82, (Portion of Embankment and Partial Excavation of Spillway). This contract was terminated following the slide of a portion of the embankment. Excavations to final grade and construction of the spillway structure were performed under Contract No. DA-41-443-CIVENG-61-693, (Spillway). Work commenced on 17 April 1961. Work was suspended in October 1961 at the time of the embankment slide. Because of the slide, it was determined that a reevaluation of the spillway foundation should be made. Investigations for this reevaluation consisted of drilling five large diameter inspection holes evenly spaced across the intake channel 10 feet upstream from the end of the weir excavation, and ten 6-inch diameter core holes at various locations in the structural foundation. Additional laboratory testing was performed on various segments of the foundation materials. As a result of this re-evaluation, the mass concrete section of the spillway was redesigned and work on the project

was allowed to continue in 1962. Construction of the spillway was completed 26 April 1964.

b. Character of the Foundation.

(1) Overburden. The only portions of the structure that were in contact with overburden materials were the upper part of the nonoverflow sections. Overburden materials consisted of alluvium and reworked fill from gravel mining operations. The gravel pits, which covered 10 percent of the area, were restricted to the intake channel and structure foundation areas and the upper portion of the discharge channel. The material comprising the pits consisted chiefly of a random mixture of clay, gravel and conglomerate fragments. All the reworked gravel pit material was placed in the waste disposal berm. The alluvium varied from 15 feet in thickness near the structure to 50 feet in thickness in the downstream portion of the discharge channel. Throughout the area of the intake channel, the structure, and the upstream portion of the discharge channel, the alluvium consisted of about 10 feet of lean clay or sandy clay overlying 4 to 10 feet of sandy gravel, clayey gravel and conglomerate. That portion of the discharge channel located beyond 1,600 feet right of the embankment axis (the downstream portion of the discharge channel) had a considerable thickness of overbank deposits from the Bosque River. At a point 1,600 feet right of the embankment axis, the bedrock surface dropped into the deep valley entrenchment where about 40 feet of lean and fat clays covered a thin (2 to 4 feet thick) clayey gravel bed. The bedrock surface was below the maximum discharge channel excavation through most of this area.

(2) Bedrock. The bedrock material throughout the spillway area was Del Rio Shale (See Plate 39). The lowest reaches of the stilling basin floor were located in the gradational zone between the Del Rio Shale and the underlying Georgetown Limestone. The Del Rio is characterized by its massiveness (lack of bedding planes and stratification). With the exception of two highly calcareous zones located slightly below the middle of the formation, indications of bedding were essentially absent. Very slight color changes could be seen at a distance in the massive shale, suggesting a bedding plane, but close examination usually showed that these changes were so gradational that no individual horizons could be located. Occasional lenses or pockets of slightly softer, dark clay shale were recognized in core samples, but could not be correlated or recognized in the excavations.

Occasional thin, lenticular inclusions of limestone and clay ironstone were encountered but were not sufficiently abundant to cause any anomalies in the loading characteristics of the shale. No significant faulting was encountered although some slickensided surfaces were noticeable. Jointing was the most significant structural condition involved in the spillway foundation. Within the areas investigated, the joints were developed in a conjugate system, with one set of joints approximately parallel to the axis of the weir section and the other approximately normal to it. Many of these joints were open enough to permit encrustations of ferric oxide to depths of 40± feet below the top of rock. Whenever these joints were encountered during

drilling, they allowed the drilling circulation to vent to the surface. Grouting was considered in some cases, but later deemed unnecessary. Joints caused considerable sloughing along vertical faces. Some mechanical deterioration occurred where exposed shale surfaces were allowed to dehydrate. This drying process ordinarily penetrated less than 2 feet before a protective mantle developed. When weathering of this type occurred, such as along the chute and stilling basin training walls, it was necessary to over excavate appropriately. Weathering in the foundation was of minor consequence. The basic excavation removed all weathered primary material from the foundation with the exception of occasional slightly weathered bands along joints.

(3) Dewatering. Ground water was encountered at the overburden-bedrock contact but was easily handled by ditching the side slopes and draining into the spillway discharge channel. A cofferdam placed across the discharge channel below the structure area prevented the Brazos or Bosque flood waters from backing up the discharge channel into the stilling basin. Ground-water seeps, rain water, and excess construction water were pumped out of the excavation. Occasional small sumps or catchment basins were dug to collect water in the excavation. Rain water posed the greatest problem. After heavy rains, work was often held up in the stilling basin to dewater. At times, several days were needed to accomplish the drainage. Usually, 2- or 3-inch centrifugal pumps were used in the dewatering operations.

c. Foundation Excavation.

(1) Preliminary excavation included the spillway intake and discharge channels, and partial excavation of the structural foundation and stilling basin. All impervious materials in the overburden were used in the embankment. Pervious materials were placed in the waste disposal berm on the downstream riverward side of the stilling basin and discharge channel. Most overburden excavation was performed using dozers and scrapers. A minor amount of the tougher clays and conglomerates required ripping, and one large conglomerate lens in the intake channel required blasting.

Bedrock excavations were considerably more involved. Although most of the shale could be easily ripped, it was necessary to accomplish some controlled blasting. Blasting was generally restricted to the lower portions of the discharge channel excavation and was done only when conditions required it. No blasting was allowed within 10 feet of finished grade. Suitable shale materials were used in the random and shale fill sections of the embankment (Station 34+00 and 80+90). All unsuable materials were placed in the waste disposal berm on the downstream riverward side of the stilling basin or in designated waste disposal areas located upstream from the spillway structure.

(2) Final excavation to foundation grade was performed under the conditions of the contract for construction of the spillway. Most vertical or near vertical faces were precut with a coal saw. The

shale was then broken up by ripping and loaded out with a track-type end loader. Fine grading was done by hand. Over excavation was moderate, generally arising from over penetration by ripper teeth or spalling along cut faces. All loose or questionable material was removed prior to placement of protective membranes. Because of the spillway design, a minor construction problem developed during excavation of the upstream footing for the redesigned portion of the structure. The design required that a shale face at the back of the structure be exposed for an extended period of time before being backfilled. This face was cut at a 4V on 1H slope, and required a protective asphalt membrane. However, because of the extended period of exposure, the face slaked and spalled badly, even though protected repeatedly with asphalt sealer. The spalling posed a constant danger to the workmen and also resulted in an extraordinary amount of cleanup, especially when shale particles would fall into concrete pours.

d. Foundation Treatment and Preparation.

(1) No dental work or foundation grouting was necessary in the spillway foundation. Minor excavations were made as required at the time protective concrete membranes were placed.

(2) Foundation treatment consisted of installation of foundation drain holes and anchor bars (See Plate 10). Anchor bars were divided into three groups. Two groups were set in the training walls and a third group in the apron and stilling basin floor. All anchor bars were grouted into 4-inch diameter holes. The first group

placed in the training walls consisted of 68 (34 in each wall) No. 14 bars, 60 feet long, grouted 31 feet into shale and extending 29 feet into the base slab of the wall. The holes for these bars were drilled on a 4V on 1H slope into the foundation from elevation 390. Individual spacing varied from 5-foot centers at the apron end of the training wall to 8-foot centers for the last six bars at the downstream end of the training wall. The second group of bars used in the training walls consisted of 110 (55 in each wall) No. 11 bars, 40 feet long, grouted 37.5 feet into shale. The holes for these bars were drilled on a 1V on 4H slope into the foundation. Spacing was on 8-foot centers in two lines at elevation 376 and 384 on the excavation face. The third group of anchors consisted of 1,925 No. 11 bars grouted 10 feet into shale at an angle perpendicular to the excavation grade surface. Spacing was a nominal 10 feet on centers, varying slightly to miss all contraction joints. Five pull-out tests were performed. Four tests yielded at the ultimate tensile strength of the bar, and the fifth test failed by pulling free of the hole. The cause of this failure was attributed to an improper grouting job.

Drain holes to intercept and relieve uplift pressures were installed in six different series. The holes varied from 3 to 6 inches in diameter and were lined with perforated polyvinylchloride pipe. All holes were drilled by rotary or rotary-percussion type tools, were thoroughly washed and cleaned, and were jetted dry before the liners were installed. The location and details of the drain holes, in the order of their occurrence (upstream to downstream), are as follows:

(a) One hundred and forty-one holes were drilled from the drainage gallery, in line, on 5-foot centers. The holes were angled upstream on a rising 1V on 20H slope from elevation 410.5 on the drainage gallery wall, 60 feet into shale. The 4-inch diameter holes were lined with 2-inch O.D. perforated polyvinylchloride pipe. No filter material was placed in the holes. During drilling operations it was noted that some of the holes vented to the surface upstream from the structure. Air from the drilling operations apparently vented along vertical joints. After installation, most of the drains made slight amounts of water, but most of this water was stopped by placement of impervious fill over the upstream portion of the weir structure. No significant increase in water production was noted from these drains after completion of dam and impoundment of water.

(b) One hundred and forty-one drain holes were drilled vertically from the floor of the drainage gallery. The holes were located in-line, on 5-foot centers. Each hole was 6 inches in diameter and extended 30 feet into shale (top of the Georgetown Limestone). A 2-inch diameter polyvinylchloride liner was centered in each hole, and the annular space between the hole wall and the liner was packed with graded sand filter material. The total inflow (flow through drains and leakage through joints in the structure) into the spillway gallery is being measured. Gallery inflow and reservoir elevation plotted versus time are shown on Plate 42. There is a general correlation between gallery inflow and reservoir elevation. Piezometers installed in the foundation at the base of the spillway structure show that the seepage pressures are relieved by the drains.

(c) One hundred and forty-one drain holes were drilled along the apron at the original location of the weir section. When the weir was redesigned and relocated, these drains were left in place and extended to drain from the surface of the apron. The holes had been drilled 40 degrees from vertical and were intended to intercept the floor of the originally designed drainage gallery, spillway Station 124+50. Each hole was 4 inches in diameter, extended 45 feet into shale, and was lined with 3½-inch O.I. perforated polyvinylchloride pipe. No water discharged from the drains before impoundment. Since impoundment many drains are flowing and appear to be functioning normally.

(d) Thirty-four drain holes were drilled 40 degrees from vertical on 20-foot centers across the apron. The holes were 4-inch diameter and intercepted the apron surface at Station 125+30. Each hole was drilled 30 feet into shale and lined with 3½-inch O.D. polyvinylchloride pipe. The holes filled with water after installation, but production was very slight.

(e) Seven hundred and fourteen drain holes were placed in the chute apron and stilling basin floor. These holes were located to include 21 lines of 34 holes each, all on nominal 20-foot centers. Minor variations were made to miss contraction joints. All holes were 4 inches in diameter and were drilled 10 feet into shale at a 30 degree angle from vertical. Each hole was lined with 3½-inch O.D. perforated, polyvinylchloride pipe. Many of the holes in the

lower part of the apron and the stilling basin made a minor amount of water after completion, but no significant production was noted. Holes in the stilling basin are presently submerged and cannot be visually observed.

(f) In addition to the vertical or angle drains, two rows of 15 each horizontal drains were drilled 33 feet into the shale. The horizontal drains were all 3-inch diameter holes, spaced along the walls on 18-foot centers, with their invert at elevation 383. No liner was placed in the holes. Several of these drains made a slight to moderate amount of water. All drains, except groups described in paragraphs a and f, were drilled through the protective concrete membrane. Groups a and f drains were drilled prior to placement of concrete at their respective faces. All drains appear to be functioning properly in accordance with their design.

(3) Surface Treatment. Specifications called for placement of 6-inches of protective concrete on all floor areas and slopes and pneumatically placed asphalt on all vertical or near vertical faces. The final grade of the chute and stilling basin walls, which were cut with a coal saw, were covered with pneumatically placed asphalt as soon as practical after exposure. The asphalt treatment was repeated as necessary until the wall was completed. Portions of the end sill and weir side walls were also cut with a coal saw. The walls of the redesigned weir section were fine-graded by hand. All vertical and near vertical walls in the weir foundation were treated

repeatedly with asphalt while they were exposed. The floors and chute sections were hand graded, cleaned, and approved before being covered by a 6-inch protective concrete membrane. Over excavation in the floors and chute was backfilled at the time the protective slabs were poured. Protective slabs were generally placed in 8 or 12 by 30-foot sections. Upon completion of the grading, each section to receive a protective membrane was throughly inspected by a Government representative and covered by a wet tarpaulin until the membrane or slab was placed. If the section was not approved, it was kept moist until the deficiency was corrected. Because the Contractor was delayed by a shutdown during construction, some of the vertical faces along the training walls were exposed much longer than originally planned. In places the deterioration was sufficient to require change orders to correct the situation. Remedial repair usually consisted of placing additional anchors, some of which exceeded 50 feet in length. A moderate amount of over excavation was also required. In all cases, loose or weathered material was removed before placement of the structural units.

e. Spillway Instrumentation. Twelve piezometers were installed at various locations in the foundation as shown on Plate 41. Each piezometer consisted of a 6-inch diameter porous concrete tip connected to a 1/2-inch O.D. stainless steel riser. It is believed that this was the first installation of prefabricated porous concrete piezometer tips. The tips were fabricated and the piezometers were installed by Fort Worth District government crews. Each tip was

imbedded 1.5 feet into the foundation at their respective locations. Two sets of piezometers were located under the weir section in pier monoliths "O" and "N". Each set consists of six piezometers; one located at the top and one at the bottom of the back wall of the mass concrete weir section; one directly under the drainage gallery 5 feet downstream from the bottom of the back wall; one 30 feet downstream from the bottom of the back wall; one 9 feet downstream from the centerline of the weir; and one 43 feet downstream from the centerline of the weir. Ten of the piezometers had their risers extended through the concrete to the roadway over the top of the spillway. The other two piezometers extend into the drainage gallery.

4-03. Embankment Foundation.

a. General. Geotechnical personnel were not assig[illegible] to the project until after the embankment slide. At that time they became responsib[illegible] for approval of foundation in the embankment slide area and from Stations 0+00 to 34+00. Portions of the embankment foundation excavated prior to the embankment slide were inspected and approved by project construction personnel. The embankment foundation excavation and treatment is discussed in three segments: the dike segment, which extends northwestward from the left nonoverflow section of the spillway for a distance of about 9,000 feet; the embankment segment, which extends from Station 34+00 to the right nonoverflow section of the spillway and the closure segment which extends from Station 0+00 on the right abutment to Station 34+00.

b. Dike Segment. Foundation preparation for this segment consisted of stripping sod, topsoil and organic material from the embankment foundation area and excavating a 5-foot deep inspection trench at the centerline of the dam. The trench had a 10-foot bottom width with 1V on 1H side slopes. Construction records make no mention of excavating bedrock or encountering unusual conditions in the foundation. The alluvial overburden averages about 25 feet in thickness in the area and is underlain by calcareous clay shale of the Del Rio Formation. The occurrence of a gravel bed in the overburden under a portion of the dike segment may allow a minor amount of seepage. However, any water seeping through the ground will enter the normal ground water aquifer in the downstream alluvium or will discharge into the excavation for Borrow Area E which is graded to drain into the spillway discharge channel.

c. Embankment Segment.

(1) Foundation Preparation. Original foundation preparation for this segment consisted of stripping the foundation area of all vegetation and organic materials and excavation of a cutoff trench to bedrock at the centerline of the dam. Geotechnical personnel were not assigned to the project during the embankment segment foundation preparation. A review of records and conversation with construction personnel indicated that operations were considered normal. The cutoff trench excavation was accomplished with dozers and scrapers and was 20 feet wide at the base with 1V on 2H side slopes. In many places ground water caused problems during trench excavations. To establish

good bond between bedrock and impervious backfill, the trench was widened and the water sealed off at the sides with clayey material. The trench bottom was then cleaned by dozers pushing a header bank. The trench was backfilled with impervious material as soon after approval of the trench as possible to preclude any water reentering the trench behind the header bank. No mention was made of the fault zones which were crossed by the trench at approximate Stations 51+50 and 58+40. Before the slide it was thought that the shale on which the trench was bottomed was all Eagle Ford. However, post-slide investigations revealed that from approximate Station 58+40 to the spillway the trench was founded on the Del Rio Shale, from approximate Stations 51+50 to 58+40 on the Pepper Shale and from 34+00 to 51+50 on the Eagle Ford Shale. The embankment slide which ultimately covered an approximate area between 50+00 and 65+00 reflected the movement which occurred in the area bounded by the faults and underlain by the Pepper Formation (i.e., approximate Stations 51+50 to 58+40). Post-slide investigations indicated that a redesign of a portion of the embankment was necessary and that treatment would be required for the foundation in the slide area. Remedial measures included degradation of the embankment in the slide area to elevation 450, backfilling the cracks in the embankment below elevation 450 with a sanded grout slurry, and addition of massive random fill berms both upstream and downstream. Additionally, trenches were excavated along the fault lines both upstream and downstream from the embankment toes to the outer end of the berms, and the bond between the foundation and the cutoff trench and the remaining Pepper Shale

under the axis was tested and grouted if necessary. The excavation of trenches allowed a thorough inspection of the faults both upstream and downstream from the embankment. Fault zones in the upstream trenches were test grouted and found to be tight. The trenches were then back-filled with impervious material. In the downstream area, a system of vertical 18-inch diameter, sand filled drains was constructed along the faults and integrated with a 5-foot thick horizontal pervious blanket that covered the floor of the trenches.

Smaller berms were also added to the portion of the embankment from Station 34+00 to the interception with the slide repair section. These berms were added as an extra precaution to improve the stability of the embankment. No additional foundation treatment was required in the area.

(2) Embankment and Foundation Remedial Treatment. Remedial treatment in the slide area consisted of grouting the embankment section which would be left in place and become a part of the redesigned embankment, grouting the foundation and treatment of the upstream and downstream fault zones which would be covered by berms.

(a) Embankment Grouting. Grouting of the embankment was initiated in June 1962 with the crest at about elevation 470. However, no grouting was planned above elevation 450, the point to which the embankment was degraded before final reconstruction. Grout was injected under gravity pressure only. Solids in the grout mix consisted of 3.7 parts fine sand to 1 part cement by volume with 5 per-

cent bentonite by weight of sand and 1 to 2 percent fluidifier by weight of cement. Grout was injected both through open cracks in the embankment and drilled holes. Since most of the open cracks were on the downstream side of the slide, grouting was initiated at the downstream toe and proceeded upslope and upstation. Locations of grout holes are shown on Plate 56.

The drilling equipment was not available at the start of the job, so grouting in cracks actually proceeded higher than the first line of drill holes. No problems arose from working higher on the embankment, so the work was allowed to progress up to approximate elevation 470 without any restraint. Drilling did not intercept as many voids as was originally anticipated. Although many holes had 100 percent water loss near the surface, it was determined that most loss was restricted to the loose, d :urbed material in the top 6 feet of the holes. Water was always lost in the drainage blanket when it was intercepted. Although many holes penetrated the drainage blanket, no voids were intercepted below it. The drainage blanket material was exposed to the grout many times without an appreciable take. After the embankment was excavated, examination showed that grout never pene -ated more than 6 inches into the drainage blanket. During drilling erations, attempts were made to limit the use of water, but water losses were considerable; however, no deleterious effects to the embankment were noted from excess water. Some holes penetrated into the overburden, downstream from the cutoff trench. In most cases water was lost in the overburden, but grout takes were small. Occasionally,

grout vented downslope from the point of injection, but this was not a major problem.

The several fixed movement points which had been established throughout the slide area were checked periodically during the grouting operations. No point movements were noted until 19 July 1962. On that date, a general movement of about 2.0-foot horizontally occurred in the area between Stations 52+00 and 60+00, from about 130 feet downstream of the centerline to the toe of the embankment. This movement was closely observed over the weekend of 20-23 July, when operations had ceased. About 40,000 cubic yards of embankment materials were removed. No further discernible movement was noted, so the grouting contractor was allowed to continue. It was concluded that the movement of 18 July was a readjustment of individual blocks within the slide area, and not a general movement along the slide plane. Some of the points in question actually started returning upstream, which would indicate a wobbling of the blocks rather than actual slipping. Another such movement in the area of Station 55+00 to 58+00, 100 to 200 feet downstream, was noted on 24 July, and new cracking in individual blocks was also noted in this area. The cracking apparently was caused by readjustment of the blocks.

Contract grouting was halted on 1 August 1962 because of a major contract change providing for remedial treatment of the faults. A definite movement of the entire mass took place on 19-20 August, almost three (3) weeks after grouting operations ceased. Since the remedial

work would encompass several months time and it was considered detrimental to add load to the embankment while unloading the toe in the slide area, the grouting contract was terminated.

Grouting operations were resumed on 5 January 1963 after the embankment was degraded to elevation 450 and were completed by April 1963. This phase of the grouting program was accomplished using hired labor forces. When the excavation reached elevation 450 and the hired labor forces started grouting, no cracks were visible; consequently, all points were drilled in and very few takes were encountered. The insertion points were drilled on a grid pattern, with additional points added where grout takes were large or where takes were suspected. Only two takes exceeded 100 cubic feet.

(b) Embankment and Foundation Grouting. During July and August 1963, after the embankment in the slide area was reconstructed to elevation 475 on the centerline, a test grouting program was completed along the centerline between Stations 51+00 and 59+00. The purpose of this program was to test and grout if necessary, the fill-shale contact and the Pepper Shale under the embankment. All the primary grout holes were drilled through the Pepper-Del Rio contact.

Drilling, testing, and grouting were performed by Government equipment and hired labor. Holes were drilled to their full depth with 3 1/2-inch fishtail bits. Unless water loss occurred, holes were drilled to their full depth in a single operation and pressure tested twice, once at the top of bedrock and once above the top of bedrock,

to check the fill shale contact. Original plans were to use gravity flow only, but the character of the fill caused the holes to block off, so moderate injection pressures were used. No significant voids were intercepted. However, numerous water losses occurred during drilling and pressure testing. These losses can probably be attributed to interception of open, ungrouted cracks in the embankment below elevation 450 that formed during the failure and were missed during crack slurry grouting operations. One hole at 53+10 on the centerline took 19.5 sacks of cement in the fill. Other grouting attempts in the fill were insignificant. One hole was abandoned and offset because it was venting into piezometer P-33C. Occasional minor takes were encountered at the fill shale contact. A maximum take for a single hole occurred over the North Fault at Station 58+35 where 75 sacks were injected. No grout takes occurred in the foundation. Pressure tests up to 25 psi were conducted in the foundation with no appreciable water losses.

(c) Remedial Treatment on Faults Upstream From Centerline. Excavation of trenches that uncovered the upstream faults are shown on Plate 53. The initial excavation consisted of a drainage ditch to dispose of surface water. This ditch ran parallel to the embankment, approximately 500 feet left of centerline between Station 55+50 to Station 27+00. A convergent tear fault between the north and south faults was exposed at Station 50+95 in this drainage ditch. At the time the drainage ditch was excavated, the location of the south fault had not been made; consequently, initial remedial work was done

on the convergent tear. Since the upstream remedial treatment was to be grouted, and since previous work indicated that most grout takes would be restricted to the hanging wall side of the faults, the upstream trenches were excavated so that the hanging wall side of each fault was best exposed. The original intent was to expose 5 feet of the footwall side, the brecciated zone, and 90 feet of the hanging wall side. This was accomplished on the convergent tear between the north and south faults, but not on the north fault. Excavation on the north fault exposed the fault at three places along the upstation wall of the trench. A cross trench at 800 feet left of centerline was connected to the upstream end of each trench. The Eagle Ford was exposed along the entire length of the cross trench. Most of the excavation was done by scrapers. A drag-line was needed to remove soft, saturated materials along the drainage ditch and the north fault trench. It was necessary to rip the Eagle Ford strata in the drainage ditch. Excavation of the north fault trench was performed intermittently prior to and during the north fault test grouting. It was originally anticipated that little test grouting would be needed on the north fault. For this reason, only three grout holes were located in the first available portion of the excavation, at about 650 feet upstream from centerline. Pressure testing on these holes indicated a need for more extensive work. This work consisted of test grouting operations which were moved as close to the upstream toe of the dam as possible, so that they might be incorporated into a future grout curtain. After the test grouting was completed, the north fault trace was exposed at

the upstream end of the trench for mapping purposes. Foundation treatment in the upstream fault excavations occurred in two phases: (1) A test grout program to determine the need for subsurface grouting in the upstream area from the north fault to the south fault, and (2) The blanketing of the fault traces at the top of rock with impervious materials. The test grout program was divided into two parts: (1) The construction of a series of grout plots on what, at that time, was considered as the south fault, and (2) The construction of a grout curtain across the north fault. The test program was performed by hired labor government forces. All grout mixes consisted of Portland cement and water placed at varying water-cement ratios as foundation conditions dictated. The location and results of the test program are shown on Plates 55 and 57 thru 59.

(d) Remedial Treatment on Faults Downstream From Centerline. The north fault was encountered in a downstream drain ditch, and was followed easily during excavation of the main trench. The position of the trench was altered slightly from the original plan so that inplace berm material would not have to be removed. The floor and walls of the north fault trench were mapped by plane table methods, and are shown in plan on Plate 51. The overburden was mapped in three categories: alluvial deposits, alluvial gravels, and reworked backfill material. Much of the area had been mined for gravel, and the old pits were filled with random material. The floor of the north fault trench was supported by both the Pepper and the Del Rio, separated by the fault. The Del Rio, which occurs on the hanging

wall side of the fault, was moderately weathered. Little distortion was noted in the Del Rio near the fault, although the two shales were throughly smeared and mixed at the fault planes. A brecciated zone on the hanging wall side of the fault at the downstream end of the trench showed considerable intermixing of the shales for about 3 feet from the fault. The Pepper slaked rapidly on exposure, and there was evidence of local rebounding in the Pepper after unloading. Horizontal partings were numerous in the top 6 inches of the exposed surface, and many irregular, joint-like vertical partings were evident during excavation. Most of this disturbed shale was removed during final cleanup of the trenches. True vertical joints were uncommon, or unrecognized, because the surface was disturbed. Weathering was not extensive in the Pepper Shale. A spring (See Note 2, Plate 51) was encountered in the downstream side of the old north inspection trench. The spring flowed approximately 3 gpm and was apparently producing from bedrock. The drainage blanket underlying the embankment was well exposed in the upstream end of the trench.

Excavation of the south fault trench was started by cutting across the downstream end, after which the proper dimensions were laid out, and the trench was stripped toward the embankment. The floor and walls of the south fault excavation were mapped by plane table methods, and are shown in plan on Plate 52. Many features of slide movement were observed. The drainage blanket, underlying the embankment, was highly disturbed and absent across most of the up-stream end of the trench. Two pressure ridges in the Pepper Shale were noted.

Both ridges entered the trench at the upstream end where the fault passed under the embankment. The first ridge left the trench at the point where an earlier inspection trench intercepted the left (west) wall. The Pepper Shale was highly distorted and actually thrust over the Eagle Ford adjacent to the fault. The second pressure ridge left the trench near the downstream end of the left (west) wall. This second ridge evidently marked the edge of the slide. During movement that took place in August 1962 a single crack was observed starting at the upstream end of the trench and paralleling the fault for about 200 feet. Actual horizontal movement along this crack amounted to about 1¼ inches. At about 200 feet from the upstream wall, the crack spread into a zone of heaving. The horizontal movement was transformed into vertical movement, and the floor was heaved as much as 4 inches vertically. This movement was marked by an upward warping rather than by overthrusting. The heaving followed the second pressure ridge closely and was traced for about 400 feet across the ground surface between the two fault excavations. The trench floor had not been cleaned sufficiently to permit observation of movement at the time it occurred. The overburden was reworked throughout the sidewalls of the trench. Alluvial deposits were encountered at the downstream end and extended a considerable distance downstream from the end of the trench.

The trench floor was composed of Pepper Shale on the footwall side of the fault and Eagle Ford Shale on the hanging wall side. The Pepper Shale showed very little drag-warping, although it was highly brecciated for approximately 20 feet adjacent to the fault. The Eagle

Ford showed strong drag-warping and little, if any, brecciation associated with the fault (See sections, Plate 54). Only minor weathering was evident on the bedrock surface in the south trench, probably because much of the weathered shale was scraped off during the gravel mining operations. Considerable oxidation was noted along the exposed bentonites. Rebounding in the Pepper Shale was less evident in the south trench than in the north trench. This was attributed to the relief of compressive pressures after the slide. Very little ground water was encountered in the excavations. Construction equipment and weathering disturbed the excavated surface of the Pepper Shale considerably and it was necessary to remove an additional foot of material during the final cleanup operations. Treatment along the downstream faults consisted of seventy-seven (77) 18-inch diameter sand backfilled drain holes. For location see Plates 51 and 52. As shown on this drawing, 39 drains, ranging in depth from 41.5 feet to 103.5 feet were installed in the north fault. The drains installed on the north side of the fault or in the upthrown block were drilled to the Del Rio/Georgetown contact. The drains located on the south side of the fault or downthrown block were drilled to the Pepper/Del Rio contact or, if in the fault, 10 feet into the Del Rio Shale. Fourteen of these drains made water during drilling or prior to backfilling. On the south fault, 38 drains were installed, ranging in depth from 53.5 feet to 82.0 feet. The drains located on the north side of the fault, or upthrown block, were drilled to the Pepper/Del Rio contact. The drains located on the south side of the fault, or downthrown block,

were drilled to the fault. Twenty-six of these drains made water during drilling or after drilling, prior to backfilling. All water production was from bedrock. No significant wall smear was noted in the holes that were drilled dry. The holes that made water during drilling developed some wall smear, on occasions. When this condition occurred, the holes were bailed and cleaned. The holes were then backfilled with concrete sand. After the drain holes were backfilled, final cleanup was performed on the excavation with a motor patrol. The fault trenches were backfilled with 5 feet of free draining gravel prior to placing compacted impervious fill.

d. The Embankment Closure Segment. This section includes the portion of the embankment from the right abutment to Station 34+00. Within this interval the embankment varies from 110 feet to 140 feet in height, measured from the cutoff trench excavations. The discussions concerning the closure section are divided into three parts: (1) The right abutment from Station -0+17 to Station 7+00; (2) the river channel, from Stations 7+00 to 21+00, and (3) Station 21+00 to Station 34+00.

(1) Station -0+17 to Station 7+00. Exploration during the spring of 1963 indicated that a large slump block existed on the right abutment. The block included an area controlled by a shear plane that dipped near vertically at Station 0+48 and extended riverward to Station 3+25, where it toed out into the alluvial valley fill (See Plates 11, 34, 37 and 38). Both the Austin Chalk and the Eagle

Ford Shale were involved in the slump. The contract plans for the right abutment required a sloping excavation between Station 0+00 (elevation 505) and Station 0+48 (elevation 409), which transitioned from a 20-foot base at elevation 505 to a 30-foot base at elevation 409. From elevation 409, the excavation was to follow the top of firm shale down to elevation 368 at Station 3+25. At Station 3+25 the elevation was to daylight on a 10V on 1H slope. The side slopes of the excavation were to transition from a 1V on 2H slope at Station 0+00 to a 1.5V on 1H slope at Station 0+48 (See Plate 23). However, problems were encountered from the start of construction. One such problem occurred on the downstream side slope which became inordinately long and steep because of the angle at which the embankment encountered the abutment. To add stability to this problem area, the slope of the transition was decreased by benching, bringing the toe of the slope out to approximate Station 0+80 at a point 60 feet right of centerline. Excavations on the upstream slope also encountered unstable rock but in this case extensive rock bolting was sufficient to control over-breakage. During the treatment of the right abutment, an inspection was made by personnel of the Office, Chief of Engineers. The results of this inspection dictated that only the slumped portion of the Austin Chalk be removed from the abutment. To accomplish this, it was necessary to excavate to elevation 393 between Station 0+79 and Station 2+92 and then slope to elevation 368 at Station 3+00. For profile of trench see Plate 23. The condition of the chalk below elevation 450 on the face of the excavation was relatively sound and

required only scaling and removal of minor overhangs (See Photos 26 and 27. On the side slopes, the loose material and overhangs were removed as much as practical, and the fill tamped in tight with hand equipment. The placement of rock bolts and protective wire mesh undoubtedly restricted the amount of sloughing, and only minor quantities of loose chalk occurred under the mesh. Very little of the mesh stretched or tightened during the time it was in place. All mesh and bolts were removed or cut as the fill was brought up.

After excavation and partial backfill of the right abutment tie-in (embankment constructed to elevation 474) a limited grouting program was performed by Government hired labor forces. The grouted section, which consisted of 25 angle holes, included the wedge of chalk from Station 0+48 to -0+17, elevation 408. This program was initiated as a three-row grout curtain with all holes penetrating into the Eagle Ford Shale (bottom of grout curtain at elevation 408). All primary holes were on approximately 10-foot centers (See Plate 37). Grouting was performed in stages where practical. Low to moderate injection pressures were used, varying with the depth of the hole. All holes reached refusal before 250 sacks were placed. Occasional communication between holes was noted, and in each case a packer was set in the leaking hole and grouting continued to refusal. The center grout hole row was split spaced with a secondary series of holes where necessary. During the grouting program two holes were cored to check the effectiveness of the grouting. Core from these holes indicated good grout coverage and an excellent bond between the grout and the chalk. A total of 25 holes

were drilled and 2240 sacks of cement were placed, of which 1928 sacks were injected into the chalk, and 312 sacks were placed by gravity flow at the chalk/fill contact or used in backfilling the holes. The grout curtain was originally planned to extend back into the abutment to Station -2+50, but the results of the referenced grouting program indicated that grouting was not necessary beyond station -0+17.

(2) Station 7+00 to 21+00. Prior to commencement of embankment work between Station 7+00 to Station 21+00 (river channel closure section) the main embankment had been constructed to elevation 455 or higher; the spillway and outlet works had been completed, a commitment had been made to complete the embankment to full height in the area of the slide, and an impervious core trench had been excavated along the axis of the closure section between Station 21+00 and the river. Considerable ground water was encountered during the trench excavation and it was necessary to widen the trench and flatten the slopes in order to lay in an impervious blanket and seal the water producing gravels. After the seal was accomplished the saturated material at the bottom of the trench was pushed riverward, and a clay header bank was formed to preclude the entry of river water. The header bank was then pushed riverward, followed by backfilling operation, which eventually advanced to the river bank.

Initial diversion of the river was started on 27 July 1964. Prior to that time the contractor had built the ends of a cofferdam that extended out on the existing embankment berms on either side of the river. A dragline had also worked previously along the river channel, moving the saturated muck out of the river bottom.

Initial work on the closure consisted of a temporary dike approximately 50 feet upstream from the toe of the cofferdam. This dike was laid on the river muck and constructed to elevation 380 in order to allow enough freeboard to clean out the channel under the cofferdam. When the river muck had been removed the channel was cleaned and sealed with clay prior to construction of the cofferdam. Very little seepage penetrated through the temporary dike, and no water problems developed in the river channel during cleanout operations. After completion of the cofferdam, the channel was advanced in increments behind a header bank. The channel was backfilled with compacted material as soon as it was approved. In the random sections of the berms and embankment, the banks of the river were pushed into the channel and processed into the fill. In the impervious section of the embankment the bank materials were hauled upstream to the random section and processed into the fill. Channel cleaning advanced downstream from the cofferdam about 70 to 80 feet per day. As operations continued, a second ramp was placed about 600 feet right of the centerline, and the pumps were moved to the location to continue drawing water out of the channel. When the cleanup operations reached the centerline, particular care was taken to tie the impervious material from the channel into the existing core trench on the left bank of the river.

Using the above described methods, the channel was cleaned to a point 550 feet right of centerline. At that time operations were suspended for approximately 70 days to allow the contractor to apply his full work force to completing the cofferdam (elevation 430) and

bring the embankment up out of the channel. The cofferdam was completed 29 August 1964 and the final 150 feet of the river channel was cleaned in mid-October.

(3) Station 21+00 and 34+00. The embankment foundation in this area was stripped to bedrock. The outlet works at Station 24+00 was covered with fill to a depth of 3 feet upon completion of the structure. A 20-foot section at the embankment centerline was covered with select impervious material to tie into the cutoff trench. Later, as a modification to the Repair of Embankment Contract, the entire embankment foundation was excavated to bedrock in the area between the outlet works and Station 34+00. This same contract modification called for the foundation to be stripped from Station 21+00 to the outlet works to remove the loose, uncompacted tailings from gravel mining operations. Between Station 31+00 and the outlet works, a large section of conglomerate was allowed to remain adjacent to the outlet works stilling basin and the downstream portion of the conduit. It was considered stable enough to hold the embankment, and its removal could have caused possible damage to the conduit. The bedrock dipped off into the deep valley entrenchment at about Station 21+00. All of the foundation area was backfilled to natural ground level with compacted material. Selected impervious material was placed in the position of the cutoff trench. The bedrock throughout the area was the Lake Waco (Lower Eagle Ford) Shale. No structural faults were noted in the foundation. A brecciated zone was noted and mapped at about Station 32+00. Portions of the area were subjected to geological mapping when shales were exposed. Maps thus prepared are presented on Plates 35 and 36.

SECTION 5 - EMBANKMENT

5-01. Ceneral Features. The earth embankment dam is principally an impervious fill, with conventional internal drainage zones included in the original design, but modified after the slide because of the wide berms added for stability. It has a crest length of 17,141 feet (excluding the spillway length of 904 feet), a crown width of 20 feet, a maximum height above streambed of 140 feet, and a total embankment volume of 16,209,000 cubic yards. Quantities of the various types of fill materials are listed in Table 1. Typical embankment cross-sections are shown on Plate 4 (original design) and Plate 5 (as-built).

a. Original Design. The design presented in Design Memorandum No. 6, July 1958, included a main embankment section from Station 0+36 to 85+70 (spillway centerline) and a dike section from Station 85+70 to the left end. The main section included an internal inclined drainage zone below elevation 464 (the projected future conservation pool level), and a horizontal drainage blanket at the base of the dam downstream from the inclined zone. The portion of the embankment upstream from the inclined drain was compacted impervious fill and the zone downstream was compacted random materials. Shale (Del Rio Formation) from the required excavation for the spillway structure was initially included in the random zone, and after it proved to be an excellent impervious fill material, it was also placed in the central portion of the embankment between elevations 432 and 470. A cutoff trench with a base width of 20 feet was constructed through the shallow

overburden on the left floodplain. The dike section from the spillway to the left end of the dam included an upstream impervious section and a downstream random section. No drainage blanket was provided because the ground surface is above the conservation pool level, and the embankment and the overburden materials above conservation pool level are of low permeability. From elevation 445 to 510, the upstream slope of the original embankment section is covered with a 24-inch thick riprap blanket on 9 inches of crushed rock bedding.

b. Post-Slide Design. A thorough investigation and revaluation of foundation conditions was undertaken after the 1961 slide. Locations of all pertinent borings drilled are shown on Plates 11 and 12. Logs of borings with results of identification tests and engineering properties are shown on Plates 17 thru 22. The revaluation led to major modifications in the embankment design for the main portion of the dam from the right abutment to the spillway. Wide berms were added to the area of the embankment affected by the slide. The shoulders of the berms are at elevation 435, at a distance of 800 feet from the axis of the dam, upstream and downstream. The surface of the berms intersect the original embankment slopes at elevation 470. From the slide area to the right abutment, upstream and downstream berms were also added. The shoulders are at elevation 420, and are at distances of 532 feet downstream and 540 feet upstream from the axis. The surface of the berm intersects the upper embankment slope at elevation 454 downstream and 446 upstream. In the slide area and from the slide area to the spillway, there is no internal drainage blanket other than that

remaining from the original construction. From the right abutment to the slide area, a high level downstream internal drain above the berm level is provided. The horizontal portion exits at the top of the berm, and the inclined portion extends up to elevation 470. Riprap on the upstream slope of the embankment from the right abutment to the spillway extends from the top of the berm to the crest of the dam. From the spillway to the left end of the dam, the riprap conforms with the original design.

c. Slope Protection. The riprap was designed on the basis of Corps of Engineers criteria of the time (1958) for a wave height of 4.6 feet. Current criteria, established in July 1978, would require larger rock particles, and a thicker blanket. The contract plans and specifications established the 1958 design as the lower limit of sizes, with the upper limit of particle dimensions about 20 to 25 percent greater. Therefore, a gradation falling in the middle to upper part of the specified range would satisfy the current (1982) Corps of Engineers criteria. Specified gradations of riprap and bedding material and ranges as placed are shown on Plate 80.

5-02. Description of Slide

The slide that occurred in 1961 was unusual in regard to the extent of deformation and the slow rate of movement. Recognition of these factors is important in the understanding of why the slide occurred and in developing remedial design measures. Because of the unique circumstances, the slide was not considered predictable within the then existing state-of-the-art.

Original construction of the portion of the dam that slid began in October 1959. The cutoff trench had been excavated to elevation 398± and backfilled, and the embankment fill constructed at a steady rate to elevation 497 in October 1961. The design crest elevation was 510, and the original ground surface was approximately at elevation 415. The first recognition of a potential slide was on 4 October 1961, when a crack was found on the downstream slope. The crack extended parallel to the axis of the dam, approximately from Station 52 to Station 61. A subsequent review of construction records disclosed that there had been signs of movement on 17 September 1961, when stakes set for placement of riprap on the upstream slope at Station 54 + 40 were found to be 0.56 feet below the established elevation. This was suspected of being a survey error, and the stakes were reset without any investigation into the cause. On 6 October, the stakes at Station 54 + 50 were found to be 0.9. foot below the proper elevation. This suggests an average rate of vertical movement near the top of the upstream slope of 0.047 foot per day since 17 September. A few small diagonal cracks appeared at the downstream toe of the dam, over the locations of what were later determined to be two faults beneath the embankment. Before 7 October, there was no evidence of movement beyond the downstream toe, and the full extent of the sliding mass was not apparent.

On 7 October 1961, the crack on the downstream slope was about 900 feet long, parallel to the axis of the dam, and was a maximum of about one foot wide. There was no perceptible relative vertical displacement of the embankment material on opposite sides of the crack. The crack was at elevation 470, about 60 feet above the natural ground surface. An inspector reported that a bulldozer operator had seen some cracks on the downstream slope on Thursday, 28 September. Assuming that they were "shrinkage cracks", he had filled them while performing slope trimming operations. It was apparent that the cracks had opened again before Monday, 2 October, because rainfall of 1.06 inches that fell on that date had washed some embankment material into them. On 7 October, the small diagonal cracks at the downstream toe of the dam could be traced to a distance of 300 feet downstream from the toe. There was not yet any evidence of bulging at the downstream toe, as would be expected in a slide of ordinary rotational geometry. Embankment construction was halted on 9 October. On 10 October, a crack about 3 inches wide was found beneath the riprap on the upstream slope of the dam. The crack was about 700 feet long, parallel to, and about 70 feet upstream from the axis of the dam. The crack which was the upstream scarp of the sliding surface, had not been apparent earlier because of the rough surface of the 24-inch thick riprap blanket that covered it. On 12-14 October, a large number of surface reference points were placed on the dam and on the natural ground beyond the toes of the dam. Elevations were determined by differential leveling and horizontal positions were determined by measuring

offsets from a line of sight parallel to the axis of the dam. The vertical component of movement of a point near the centerline of the dam is shown on Plate 48. On 13 October, the downstream limit of the movement was found to extend about 500 feet downstream from the toe of the dam. At this time, the measured movement near the slide scarp was less than 0.1 foot, but the crest of the dam had probably subsided about 1.5 feet, as suggested by the evidence of the stakes that had been set to provide the grade for placement of riprap. As shown by Plate 48, the movement accelerated until 27 October, when the movement became slower and essentially stopped by 1 November. At cessation, the total measured horizontal movement at the slide surface on the upstream slope was 22 feet. The total measured horizontal movement of a typical point at the downstream limit of the sliding mass was 4 feet. Until 23 October, the area between the downstream toe of the dam and the downstream limit of the slide appeared to have moved as an intact horizontal plate. From 23 to 27 October, bulges and cracks appeared in this area as it buckled.

The base of the sliding mass was at elevation 370, approximately in the middle of the Pepper clay shale stratum. The location and shape of the slide surface was inferred before the movement stopped by projecting the measured surface movements of the sliding blocks into the foundation. This was later confirmed by drill holes. The sliding surface had a log spiral shape from the scarp on the upstream slope, down through the middle of the cutoff trench, becoming horizontal at elevation 370. The sliding surface rose to the ground surface at an

average distance of 770 feet from the axis of the dam. The total length of the sliding surface was about 940 feet. At the upstream toe, a slight bulge developed on the natural ground surface. It rose about 1 foot and extended about 100 feet upstream from the toe. There were no cracks connecting this movement with the slide scarp or with other cracks on the upstream slope. Two factors contributed to the difference between upstream and downstream movements: (1) The upstream slope was considerably flatter than the downstream slope, resulting in lower shear stresses in the foundation, and (2) The faults converged upstream, helping to confine and minimize the movement.

5-03. Embankment Materials.

a. Types and Sources. The soil materials for the compacted embankment zones were obtained from borrow areas downstream from the dam on the left abutment, from the area between the old Lake Waco and the new dam on the left abutment, and from upstream of the old dam on the right abutment. Del Rio shale used for compacted embankment fill was obtained from required excavation for the spillway structure. The soil from borrow ranged from clay (CL and CH) to clayey sand (SC) and clayey gravel (GC). The predominant soil was CL, which comprised about two-thirds of the available borrow soils. Standard AASHO compaction test results from borrow samples are shown on Plate 60. The typical soil used in the embankment has the following average index characteristics:

Liquid limit:	36
Plasticity index:	23

Percent fines: 75

Specific gravity: 2.64

The average index characteristics of the Del Rio shale used in the embankment were determined from the results of tests on record samples taken from the embankment during construction. During the original design, as presented in DM 6, the susceptibility of the shale to compaction was not recognized. After excavation of the Del Rio shale in the spillway area, it was apparent that this shale could be compacted into a uniform, dense impervious fill. At that time, laboratory compaction (Plate 61) and classification tests were performed on the shale. The Del Rio shale material was so well broken down and compacted in the fill that it had the appearance of a CH material, and its water content and degree of compaction was controlled as for a soil. Because it was well compacted, it was possible to cut laboratory specimens for triaxial compression and for consolidation tests with no more difficulty than for a stiff clay.

The average index characteristics of the Del Rio Shale used in the embankment are:

Liquid limit: 61

Plasticity index: 43

Percent fines: 95

Specific gravity: 2.73

Drainage blanket materials consist of crushed stone processed to provide gradations within the following limits:

Sieve Size	% by Weight Passing
Drainage Blanket "A"	
3-inch	100
1 1/2-inch	70-100
No. 4	25-60
No. 40	0-15
No. 100	0-5
Drainage Blanket "B"	
6-inch	100
3-inch	65-100
1 1/2-inch	40-60
3/4-inch	25-40
No. 4	0-15
No. 10	0-5

The major portion of the horizontal and inclined internal drainage zones was "A" material. "B" material was placed in the downstream ten feet of the horizontal blanket, providing a coarser toe drain. Specifications required that the fine-grained portions of the drainage blanket have a plasticity index of less than 4.

b. Compaction. Standard CE compaction tests were performed on samples of borrow and required spillway excavation material covering the range of materials expected to be used in the embankment. The compaction curves are shown on Plates 60 and 61. To provide a range of densities for determination of potential variations in engineering properties, samples of fine-grained soils were also compacted in the laboratory using 40 percent and 20 percent of the standard compaction effort. This produced dry densities from 93 percent and 88 percent of the maximum density defined by the Standard test, respectively. A correlation was developed between the liquid limit and the maximum dry density and the optimum water content. From these correlations, the maximum dry density and the optimum water content of the typical embankment soil were determined to be 110 pounds per cubic foot and 16 percent, respectively. The values selected as the basis for design were a dry density of 105 pounds per cubic foot (95 percent standard maximum density) and a water content of 19 percent (optimum water content plus three percentage points). Experience had previously shown these to be reasonably attainable limits for ordinary conditions of construction control.

c. Shear Strength. Consolidated-drained direct shear (S), consolidated-undrained triaxial compression (R), and unconsolidated-undrained triaxial compression tests (Q) were performed on the selected materials compacted to the range of densities mentioned above. The purpose was to establish a set of representative shear strength values and to determine potential variations from these representative values

that would result from variations in types of material and in placement water contents and densities. To relate the S strength to the "typical" soil, friction angles from the S tests were correlated with the liquid limits. To relate R and Q strengths, the results of the two types of triaxial compression tests were correlated with dry densities. Based on the correlations and the selected design density, the following embankment properties were selected for use in the original design:

S strength:	phi = 25°, c = 0.1 tsf
R strength:	phi = 12°, c = 0.4 tsf
Q strength:	phi = 3°, c = 1.0 tsf
Dry unit weight:	105 pcf
Moist unit weight:	125 pcf
Saturated unit weight:	128 pcf

After the slide, the design data were modified in recognition of the actual condition of the embankment as built. Outside the slide area the "S" strength was reduced based on judgmental conservatism, recognizing the effect of shale in the fill, which has a lower "S" angle of internal friction than the compacted clay soils. The"Q" and "R" strengths of the fill were the same for redesign as for original design because tests on samples taken from the embankment showed higher strengths than the original design tests; therefore, use of the design test data was recognized to be conservative. Outside the slide area, the following characteristics were used for redesign of the embankment:

S strength:	phi = 22°, c = 0.0 tsf
R strength:	phi = 12°, c = 0.4 tsf
Q strength:	phi = 3°, c = 1.0 tsf
Dry unit weight:	107 pcf
Moist unit weight:	130 pcf
Saturated unit weight:	130 pcf

Within the slide area, the same values were used for R and S conditions, but the Q strength values were changed to:

Above elev. 450:	phi = 0°, c = 1.0 tsf
Below elev. 450:	phi = 0°, c = 0.4 tsf

The lower value below elevation 450 was used because the fill that was left in place had been weakened by the slide movement. Strengths for various embankment and foundation conditions and materials used in redesign are summarized in Table 2.

d. Consolidation. Controlled expansion-consolidation tests were performed on samples of the selected fill materials compacted to the same range of densities used for the shear tests. The consolidation characteristics were correlated with dry densities and liquid limits. From these correlations, an e-log p curve was developed to represent the characteristics of the typical embankment soil for use in design.

e. Permeability. Based on analysis of the rate of consolidation soil compacted to a dry density of 105 pounds per cubic foot at a water content of 19 percent was estimated to be 6.0×10^{-7} cm/sec.

The permeability of the drainage blanket was estimated to be 0.2 cm/sec, based on an effective size of 0.5 mm.

5-04. Embankment Placement.

a. Compaction Criteria. All of the embankment soils were required to be compacted to at least 95 percent of the laboratory Standard maximum dry density. Based on experience, it was expected that this degree of compaction could be attained within a reasonable range of water contents. The results of the shear tests showed that this degree of compaction, with a water content of optimum plus three percentage points, would provide adequate strength for an embankment of the height planned. Originally, the Del Rio Shale fill material was not expected to be susceptible to compaction as a soil; however, initial placement of shale in the second contract (Portion of Embankment) demonstrated that it produced an excellent impervious fill.

b. Compaction Procedures. Specifications required that impervious and random soils be placed in lifts not more than 8 inches thick (loose) with a water content (after compaction) between optimum minus two percent and optimum plus three percent. The soil was compacted by eight passes of a heavy tamping roller or rubber tired roller. See photos 18 thru 25 for various types of construction equipment used and photos of fill placement operations.

In the contract for "Portion of Embankment," the Del Rio Shale fill was placed in 6-inch thick loose lifts and was compacted by four passes of a heavy tamping roller followed by two passes of a 50-ton pneumatic-tired roller. Initially the shale was placed in the portion of the dam downstream from the inclined drainage blanket, but when the two-stage compaction proved to compact it into a well knit impervious mass, it was also placed in the central portion of the dam between elevations 432 and 470. This permitted utilization of a larger volume of shale from required excavation. The placement water content range was from optimum to optimum plus three percentage points.

In the contract for "Completion of Embankment," the specifications for shale fill were the same as in the previous contract. For reasons unknown, the compaction was actually done by six passes of the tamping roller followed by one pass of the 50-ton rubber-tired roller. In this contract, the shale was placed only in the upstream and downstream berms, by administrative decision.

A summary of field construction control and record sample test results is presented in Table 3. The record samples were undisturbed samples, 7$1/2$ inches in diameter and 10 inches high, taken from the compacted fill. A suite of shear and consolidation tests was performed on these samples in the Southwestern Division Laboratory.

Drainage blanket material was placed in 12-inch loose lifts, wetted liberally by sprinkling, and then compacted by four passes of a crawler tractor having a weight of not less than 20,000 pounds. In the contract for "Portion of Embankment," the Contractor was permitted

to construct the impervious fill upstream from the inclined drainage blanket on the 1V on 1H slope, to a height of not more than 10 feet above the surface of the adjacent drainage zone. The 1V on 1H impervious slope was then graded smooth. The material removed was not permitted to contaminate the drainage zone. Drainage blanket material was then placed adjacent to the impervious face in 1-foot lifts as specified.

In the contract for "Completion of Embankment," the drainage zone was constructed a lift at a time along with the adjacent impervious and random zones. The construction surface was graded to provide drainage away from the drainage zone.

Bedding material was placed on the upstream slope by means of a "Gradall" type device, which could place and spread the material without causing segregation. There was no compaction. The crushed limestone was placed in the full 9-inch thickness (perpendicular to the slope) in a single layer.

Limestone riprap was placed on the bedding layer by means of a skip. The skip was loaded with the rock and placed on the slope, then tilted to slide the rock onto the prepared bedding layer. The rock was reworked by hand to achieve a uniform, stable surface.

c. <u>Control and Record Tests</u>. Quality assurance inspection and testing were performed by Corps of Engineers forces. Tests were performed by the project laboratory with the following average frequencies:

Water Content, Density, Classification

	Actual
Impervious fill	1 test per 1100 cu. yd.
Random fill	1 test per 2100 cu. yd.
Shale embankment fill	1 test per 1600 cu. yd.

Undisturbed record samples were taken from the compacted fill fo strength and consolidation testing by the SWD laboratory with th following frequencies:

	Actual	Target
Impervious fill:	1 test per 42,000 cu yd	1 test per 30,000 cu yd
Shale fill:	1 test per 65,000 cu yd	1 test per 30,000 cu yd

Required gradation tests on bedding material was 1 per 1000 cu yd and 1 per 5000 cu yd on riprap. The actual test frequencies is not known.

5-05. Stability Analyses.

a. General. The design strength data are presented in Table 2. In recognition of the increased understanding of the character of the foundation materials, all sections of the originally designed embankment from the right abutment to the spillway were redesigned after the 1961 slide with two (2) exceptions; (1) The rapid drawdown analysis for the portion of the upstream slope above elevation 470, and (2) all of the upstream slope from station 60+30 to the spillway. The slide area was represented by a cross-section at Station 55+00 (See plate 49). A berm was added on the upstream side between Stations 48+00 and 60+30. A berm was added downstream between Station

50+80 and 71+30 (start of existing spoil fill). From Station 5+25 to 45+00, a single section was designed, but the section was analyzed for different conditions represented by the right floodplain, the river section, and the left floodplain. The embankment slopes were flared to provide a wider contact at the right abutment. Transition sections were provided between Stations 0+00 and 5+25, and between 45+00 and 50+80. Stability analyses are shown on Plates 62 through 77. A summary of safety factors is presented in Table 4.

b. Slide Area. The slide was analyzed by use of circular arcs for active and passive zones, connected by a neutral block bounded by a horizontal plane at elevation 370 in the Pepper Shale. The location of the sliding surface was determined by exploratory borings and by projection of measurements of surface movements. Circular arcs were used because they closely matched the observed surface of sliding. Pore pressures measured after the slide movement ceased were projected backward to the conditions that were believed to exist at the beginning of the slide. Assumptions of pore pressures on the slide surface in the Pepper Shale are shown on Plate 76. Resultant forces of the active, neutral, and passive units were assumed to act horizontally. From the analysis, the effective residual shear strength of the Pepper Shale was determined to be approximately phi = 8°, c = 0.0. These values were used in the analyses of the redesigned section, giving a computed safety factor (post-construction) of 1.15. This would not ordinarily be sufficient for design purposes; however, in view of the thorough investigations of subsurface conditions, it was the opinion

of all concerned that it was adequate for the circumstances and would provide for reasonably economical construction. The slide area was not analyzed for other conventional loading conditions because of the presence of the unusually wide berms and the expectation of increase in stability with time.

c. Right Abutment to Slide Area. This section was analyzed for potential sliding planes in (a) the overburden and in the upper Eagle Ford Shale on the right floodplain, (b) the embankment in the river section, and (c) a bentonite seam in the lower Eagle Ford Shale on the left floodplain. The analyses of potential slides into the foundation materials were made by the wedge method because there were potentially weak horizontal planes in the foundation. Earth forces were conservatively assumed to act horizontally. For the steady seepage condition, the reservoir was assumed to be at elevation 500 (top of gates) and the tail water was elevation 432 (maximum design condition).

The lowest computed safety factor for the post-construction condition was 1.32. The lowest computed safety factor for the steady seepage condition was 1.58 for R-strength, and 1.64 for S-strength.

The analyses for stability under rapid drawdown and for critical pool level were made by the differential slice adaptation of the circular arc analysis. For rapid drawdown, the reservoir level was assumed to drop from elevation 500 down to elevation 455 at a rate of approximately 0.5 foot per day. For impervious fill materials, this may be considered rapid. Based on the R-strength, the computed safety factor

for this condition is 1.37 assuming a deep circle that dips to elevation 400 within the embankment in the river section and emerges on the upper part of the berm. Rapid drawdown effects were analyzed in the original design (DM #6) for a shallow circle that emerged above elevation 470, and using the S-strength factors that were considered appropriate at that time. The calculated safety factor was 1.29. If the re-evaluated S-strength were used, the safety factor would be 0.85. However, it was the opinion of the Corps of Engineers and the Board of Consultants that a shallow circle analysis of this type, approaching an infinite slope, is not appropriate for a cohesive embankment. The upper embankment slope of 1 vertical on 3 horizontal is considered adequate in light of experience.

5-06. Seepage Control.

a. Embankment Seepage. The original design provided for a horizontal drainage blanket five feet thick beneath the downstream portion of the embankment between the right abutment and the spillway. This blanket was joined to an inclined internal drain extending on a 1V on 1H slope up to elevation 464. This drainage feature had been placed in the portion of the dam in which the slide occurred. The new sections of the dam that were designed after the slide had such wide berms that no base drainage was necessary. However, a high level 3-foot thick horizontal drainage blanket was placed in the downstream portion of the fill between the right abutment and the slide area. This blanket is based at elevation 454, the top of the downstream

berm. It is joined to an inclined internal drainage zone extending on a 1V on 1H slope up to elevation 470. This provides seepage control within the normal range of reservoir fluctuations, and provides a contact drain against the steep right abutment. Because the embankment materials have very low coefficients of permeability, the quantity of through-seepage is expected to be very small.

b. Foundation Seepage. From the right abutment to the river, the embankment is on a 30- to 40-foot thick impervious blanket overlying shale. No foundation seepage control is needed, but a trench excavation was made at the foot of the abutment to remove some slumped material that may have provided a seepage channel. The embankment in the river crossing is on Eagle Ford Shale. Typical piezometer readings of fill and overburden pezometers used to monitor seepage at state 8+00 are shown on plate 43.

Between Stations 19+00 and 23+00 and Stations 25+00 and 34+00, the gravelly overburden was removed between a line 400 feet upstream and a line 300 feet downstream from the axis of the dam. This was replaced with compacted random material of low permeability. Between Station 23+00 and 25+00, the overburden had been removed and replaced with impervious fill as a part of the outlet works construction. This had been in place for more than two years when the completion contract was started. To assure that there were no cracks in the fill around the outlet conduit, a cutoff trench with a base width of 20 feet was constructed in the conduit fill between Stations 23+00 and 25+00.

From Station 34+00 to the spillway, a cutoff trench with a base width of 20 feet was excavated through the overburden to the underlying shale. Because of the location of the sliding surface, intersecting the center of the trench, the cutoff was not breached by the slide. Large diameter inspection borings disclosed an excellent bond between the impervious clay backfill and the shale in the base of the trench.

From the spillway to the left end of the dam, no foundation drainage provisions were added because of the high elevation of the ground surface (above conservation pool level) and because of the thick impervious cover. The spillway approach channel cuts through the impervious cover and through the gravelly overburden. To provide a longer seepage path beneath the embankment to the left of the spillway, the left slope of the approach channel was blanketed with a 10-foot thick impervious blanket for a distance of 400 feet upstream from the axis of the dam.

c. Fault Drainage. There was some concern about the possibility of uncontrolled seepage through the two faults that bounded the slide. Therefore, it was decided to provide controlled drainage of the faults. The overburden was removed from the area of the faults between the downstream toe of the original embankment (approximately 300 feet from the axis) and the downstream toe of the downstream berm (approximately 900 feet from the axis). The width of the trenches varied from 130 to 150 feet. In the south trench, 38 auger holes

(18-inch diameter) were drilled to depths ranging from 53.5 to 82.0 feet. In the north trench, 39 holes were drilled to depths ranging from 41.5 to 103.5 feet. The holes were filled with sand. A 5-foot thick blanket of gravel was then placed in the base of the trenches. The remainder of the trenches were filled with compacted impervious fill. Open trenches were excavated downstream of the berm to provide removal of any seepage flow.

Beneath the upstream berm, trenches were excavated along the two faults, with a cross trench joining them at the upstream end of the berm. Test grouting was performed in the trenches along the faults. After completion of the test grouting, all three trenches were filled with compacted impervious fill. The grouting was discussed in Paragraph 4-03.

SECTION 6 - INSTRUMENTATION

6-01. Original Design. The design presented in Design Memorandum 6 provided for the installation of settlement plates to measure settlement of the foundation at Stations 8+00, 18+00, and 65+00. However, it was later determined that it would not be adequately justified, so they were not installed. Had they been installed, they would not have provided any advance warning of the slide.

Given the misunderstanding of the foundation conditions and the lack of knowledge about pore pressure development in clay shales, there was no apparent justification for installing piezometers in the foundation.

6-02. Post-Slide Design. When the slide movement was recognized in October 1961, a number of surface reference points were installed in the slide area and contiguous areas. The vertical locations were determined as the slide progressed by differential levelling, and horizontal locations were determined by measuring offsets from an initially established line of sight. Measurements were made daily until the slide movement ceased, about 23 days after the reference points were established. The reference points provided valuable geometry information for analysis of the slide. These points were abandoned during excavation of the upper part of the embankment and construction of the berm in the slide area.

After the slide movement ceased and during completion of the dam, an extensive instrumentation system was established. For locations of instruments, see plates 40 and 41. The total system included:

148 foundation piezometers (7 overburden, 141 primary)

14 embankment piezometers

32 settlement plates

19 slope indicator tubes

49 surface reference points

15 reference marks on spillway

19 reference points in the outlet conduit

171 reference points on the crest of the dam

The points in the conduit are included here because they provide a measurement of deformation of the Eagle Ford foundation beneath the embankment. Instruments on the upstream side of the dam, subject to inundation by the reservoir, have been abandoned. A list of instruments currently available and in working condition is as follows:

67 foundation piezometers (4 overburden, 63 primary)

3 embankment piezometers

20 settlement plates

4 slope indicator tubes

45 surface reference points

15 reference marks on spillway

19 reference points in the outlet conduit

91 reference points on the crest of the dam

A discussion of typical results of instrument measurements is presented in Section 7.

SECTION 7 - INSTRUMENTATION DATA

7-01. Reservoir Levels.

a. To assure ample time for observation and interpretation of instrumentation data, it was desirable that the filling of the reservoir to the top of conservation pool level, elevation 455 ft msl, would be at a carefully controlled rate. The Corps and its Board of Consultants decided to try to control the pool rise in 5-foot increments between elevation 430 and elevation 455, holding each increment for 30 days while the dam was observed and the instrumentation readings were evaluated. This would allow time for remedial action if any unfavorable conditions were detected.

b. Deliberate impoundment began on 26 February 1965, and the pool rose in 5-foot increments up to elevation 440 essentially as planned. In early May, there were unusually heavy rains on the watershed, and between 8 May and 17 May, the reservoir rose to elevation 467.34, or 2.34 feet above the spillway crest. By 22 May 1965, the reservoir surface had dropped to elevation 455. There was no evidence of adverse conditions in the embankment or its foundation. Some piezometers showed a rise of 4 feet in pressure in the Pepper Shale between the faults. There were no perceptible deformations of the embankment and foundation associated with the reservoir rise or fall. Therefore, it was decided to proceed with normal operation of the reservoir at the conservation pool level, elevation 455.

c. The pool has been maintained generally within a few feet of elevation 455 over the years from 1965 to 1988. On six occasions, including the one in May 1965, the pool has exceeded elevation 465. The maximum pool level attained was elevation 470.86, in May 1968. The variations in pool level have had no apparent adverse effect on the stability of the dam.

7-02. Movements and Deformations.

a. During the 1961 slide, a large number of surface reference points were placed. They were of great value in the interpretation of the geometry of the slide, and in understanding its progressive nature. Measured movement along the slide surface ranged from 22 feet at the scarp on the upstream side of the crest to 4 feet at the downstream limit of the slide, about 770 feet downstream from the axis of the dam. The total length of the slide surface was about 940 feet. During excavation and berm construction in the slide area, all of these points were destroyed.

b. In August 1962, when the overburden was removed above the faults downstream from the dam, the sliding mass began to move again. The total additional movement observed on the shearing surface about 400 feet downstream from the axis was 0.1-foot. However, Settlement Plate K, 750 feet downstream from the axis was raised about 0.2 foot by this movement between 10 and 24 August 1962. It was quickly brought to a halt by the removal and wasting of about 40,000 cubic yards of material from the crest of the embankment. This amount of

movement would not usually be considered significant. However, in this case, the movement was steadily accelerating, and it was occurring on a failure surface where the shearing strength had been reduced to a residual (steady state) value. The initiation and cessation of the movement showed how tenuous was the degree of stability.

c. Surface reference points were considered to be the simplest, most reliable and most meaningful equipment for monitoring the stability of the embankment. After the berms were constructed in the slide area, a new grid of points was established. These were observed during completion of the dam as the fill was added from elevation 475 to 510 and during operation of the reservoir to date. The addition of fill above elevation 475 was made at a rate of a foot every three to six days. This permitted time for observation and for evaluation of the influence of the added load. As the embankment in the slide area was raised from elevation 475 to elevation 510, the horizontal movement in the downstream direction was 0.23-foot at a point 125 feet downstream from the axis of the dam, and 0.18-foot at a point 525 feet downstream from the axis. From completion of the fill in the slide area in 1964 to 1988, additional movement of the points on the surface of the berm has been about 0.10-foot downstream.

d. Vertical movement (consolidation) of the foundation in the slide area since completion of that portion of the embankment in August 1964, varies from about 2.4 feet at the axis of the dam to about 1.6 feet near the ends of the berms. The relationship between

height of fill and amount of consolidation that this implies is not valid because the measurements at the axis of the dam did not begin until after the slide. Therefore, there has undoubtedly been additional, unmeasured consolidation between the initial placement of fill in 1959 and the beginning of placement of the berms in November 1962. The observations made in the area beneath the berms, between about 300 feet and 900 feet from the axis, represents all of the consolidation that has taken place in these areas.

e. Since completion of the embankment, the points on the foundation and corresponding points on the surface of the embankment show essentially the same amount of settlement. This indicates that there has been essentially no settlement within the fill material since completion. This is considered compatible with the moderate height of the fill and the high degree of compaction.

f. At Station 35, where the embankment is founded on the Eagle Ford, the settlement at the axis of the dam is about 1.4 feet. Beneath the berms, the settlement is only 0.3 to 0.1 foot. This is a surprising differential, but may be explained by the fairly high apparent maximum past pressure of the Eagle Ford Shale. In this area, the crest of the dam is about 1.4 feet below the design crest elevation of 510 ft msl.

g. Between the right abutment and the old river channel, where the embankment is founded on up to 40 feet of overburden, the settlement of the foundation under the axis of the dam has reached 2.5 to

3.0 feet. However, the crest of the dam is essentially at or above design grade because of overbuild and construction ramping to provide access to the right abutment.

7-03. Inclinometer Tubes. Nineteen tubes were installed over the period of investigation of the slide and completion of the embankment. Many of the tubes are sufficiently damaged by either compressive or shear deformations so that they are no longer useful to identify zones of significant movement (See Photos 28 and 29). In October 1983, SI-14, at Station 54+20, 140 feet downstream from the axis, was read. The total depth of tube installed was 139.24 feet, but the instrument probe could not pass the 88-foot depth. This is about 26 feet below the top of the Pepper Shale, or approximately the elevation of the slide surface. Above this level, the tube shows a deflection of about 0.6-inch within the Pepper Shale. The deformations in 1983 were essentially the same as those observed in March 1967. In retrospect, very little useful information was derived from the inclinometer tubes. There was a problem with drift in calibration of the reading devices, and they were sent back to the factory repeatedly for repair and calibration. Although sliding connections were made between sections of the tubing, several of the tubes were crushed vertically by small amounts of consolidation of the foundation material around them. Consequently, the surface reference marks necessarily had to be depended upon to observe horizontal movement.

7-04. <u>Pore Pressure</u>.

a. During analysis of the slide and reconstruction and completion of the embankment, 148 piezometers were installed in the foundation and 14 in the embankment. Those in the fill have shown no significant excess pore pressures. Varying foundation conditions to be monitored can be divided into four major areas: (1) The spillway to the north fault, (2) The slide area between the north fault and the south fault, (3) The south fault to the river fault, and (4) The river fault to the south abutment. Within these areas are significantly different foundation materials: (1) Overburden, (2) Upper Eagle Ford, (3) Lower Eagle Ford, (4) Pepper, (5) Pepper-Del Rio contact, (6) Del Rio, and (7) Georgetown Limestone. It is apparent that the number of piezometers is not adequate for thorough coverage of all areas of interest even if all piezometers are functioning satisfactorily. Because of improper installation, corrosion of pipes, shearing of pipes, and leakage of seals, a large number of piezometers must be considered to be malfunctioning. In 1982-1983, a review and rehabilitation program resulted in the selection of 67 piezometers for continued monitoring. Because of the sparse coverage of these piezometers, it is necessary to interpret the results in terms of significant coordinated trends of a number of piezometers studied as a group. This provides a more meaningful picture of pore pressure development under the stresses caused by the construction of the embankment, the very slow dissipation of pore pressure, and the resulting increase in embankment stability during the years since completion. Typical pro-

files of piezometeric elevations at selected stations are shown on Plate 43.

b. After the 1961 slide, investigations showed development and unusual distribution of excess pore pressure in the clay shale foundation formations. Within the limits of construction under Contract No. 60-82, between Stations 35 and 80, the same height of embankment had been built on three different formations. Each formation exhibited significantly different excess pore pressure responses. Between the spillway and the north fault, the Del Rio Shale showed about 30 percent excess pore pressure under the axis of the dam. Between the north and south faults, the response at the mid-Pepper was about 80 percent of the embankment load, and at the Pepper-Del Rio contact, it was 100 percent. South of the south fault, the response was less than 5 percent in the Eagle Ford Shale.

c. Measured pore pressures were the greatest at the Pepper-Del Rio contact, and these high pressures were transmitted laterally beyond the embankment toes. A significant contributor to the high pore pressures and the transmission of pressures at the contact was the presence of the much stiffer Del Rio Shale underlying the more compressible Pepper Shale. Between the north and south faults, the pressure reduced to normal hydrostatic conditions about 1,000 feet away from the axis of the dam. At the mid-Pepper the pressure reduced to normal levels downstream within the area of fractured material that was affected by the slide movement.

d. To the south of the south fault, the excess pore pressure at the Pepper-Del Rio contact spread to distances of 1,500 to 2,600 feet from the loaded area.

e. Typical excess pore pressure gradients in the clay shale formation in the slide area are shown on Plate 44. To analyze the piezometer data and obtain more nearly correct gradients, the effect of long piezometer filter zones has to be considered. This is necessary because piezometers with long filter zones record the pressures present at the top of the filter and not necessarily at the intake or wellpoint. The solid triangles on the above referenced plate show the hydrostatic profile obtained assuming that the excess pressure head was acting at the wellpoint. The gradient shown by the open symbols shows the excess pressure heads taken at the top of each filter. Note that the shorter the filter zone, the less translation of pressure on the diagram is required. The piezometers showed a well defined gradient of excess pore pressure from the Pepper-Del Rio contact upward to the top of shale. This was contrary to conventional interpretation of stress distribution under a loaded area, which would imply lower increments of stress and lower induced pore pressure with increased depth. However, this gradient is reasonable considering that recent research by the Corps of Engineers has shown that the pore pressure induced in a clay shale (or any other earth material) is dependent more upon the relationship between the modulus of deformation perpendicular to bedding and that parallel to bedding than to depth.

f. As the embankment was completed, the pore pressures increased in the general proportions that were observed after the slide. Since completion of the dam in 1965, the overall reduction in excess pore pressure to date has been about 25 percent. Within the slide area, the reduction in the mid-Pepper has been about 20 percent beneath the axis, and about 36 percent at the end of the berm. At the Pepper-Del Rio contact, the dissipation has been about 13 percent beneath the axis and about 17 percent beneath the end of the berm. However, the measured reductions in pore pressure are based on limited and somewhat sketchy data. Many of the piezometors are nonfunctional.

7-05. Seepage Gradients.

a. A few piezometers were installed in overburden materials to measure any pressures resulting from underseepage. In the flood-plain between the old river bed and the right abutment, the piezometric levels in the overburden show a normal gradient that varies with reservoir level. The levels drop to normal ground-water elevation beneath the mid-point of the downstream berm. There is no cutoff trench between the right abutment inspection trench at about Station 7 and the river bed fill at about Station 14. Because the overburden is a soil of low permeability and because the overlying embankment has long stability berms, no special seepage control measures were constructed. The performance of the structure and the foundation appear satisfactory. Typical piezometric readings of fill and overburden piezometers used to monitor seepage are shown on Plate 43.

b. In the slide area at Station 55, the piezometers in the overburden show a very flat gradient, from 50 to 60 percent of the reservoir head immediately downstream of the cutoff trench, to tailwater level beneath the downstream half of the downstream berm. The cutoff trench was distorted by the slide movement, but was not fully breached. In spite of this disturbance and the presence of the fractured shale in the north and south fault zones, the cutoff trench appears to be 40 to 50 percent effective in reducing the seepage pressure in the overburden.

SECTION 8 - REVIEW OF STABILITY

8-01. General. A review of embankment stability was undertaken during 1984 as a part of the completion report effort. This review provided means for verification of analyses and conclusions made in 1961-1964, and for assessment of the change (increase) in stability during the 20 years since completion of the embankment in the slide area.

8-02. In-service Data.

a. Settlement plate, reference mark and inclinometer data (since completion) collectively indicate no deformations outside the range normally experienced by large earthfill embankments of similar size and geometry. However, many of the inclinometers and other instruments are nonfunctional.

b. Pore pressure data from piezometers accumulated since 1961 probably represent the single most important data available for quantitatively assessing the stability of Waco Dam. It was considered that only through analysis of these data, could a better understanding of the slide mechanism be developed. Consequently, a series of stability analyses were conducted using measured and/or projected excess pore pressures considering various combinations of embankment geometry, slide configuration, elevation, and material strengths.

8-03. Stability Analyses. The range of conditions considered in the 1984 stability study along with results are presented on Plates 64,

65, and 77. Embankment configuration considered were: 1) Embankment geometry at the time of the slide, and; 2) the reconstructed embankment section. The slide configurations considered were: 1) A "short path" which includes the pre-slide embankment and only the foundation immediately beneath it plus a minimal passive zone beyond the toe, and; 2) a "long path" which includes the fully developed slide mass extending over 700 feet from the embankment centerline as observed after movement ceased. The pore pressure assumptions were based on the re-evaluation of piezometers and the observed upward excess gradients within the Pepper Formation. "S" shear strengths were used, ranging from a residual (steady state) value to values of internal friction and cohesion that were varied dependent on the density of the shale. Of itself, the value of the "S" angle of internal friction for a given material would not usually vary with initial dry density. However, because of the manner of deposition, consolidation and rebound of the Pepper, the Atterberg limits and the insitu dry density vary with depth. There is a relationship between the Atterberg limits and the insitu dry density that provides a consistant implied relationship between insitu dry density and the angle of internal friction for the consolidated-drained (S) condition. The analyses and the assumptions on which they are based are shown on Plate 64 for conditions at the inception of the slide in 1961, on Plate 65 for conditions at the end of construction of the slide area in 1964, and on Plate 77 for conditions in 1984. All of these analyses were performed using the wedge method, assuming horizontal active and passive earth

forces. Assuming horizontal forces provides a slightly conservative estimate of the safety factor.

a. <u>Cases examined - 1961 condition</u>.

(1) Cases 1, 2, 3, 4 and 5 are based on intact strength in the Pepper that varies with in-situ density, and therefore with depth. These cases were analyzed for short slide paths, assuming that the shearing surface ended in a passive wedge at the downstream toe of the dam, and for long paths, assuming a passive wedge at the observed downstream end of the slide.

Long path cases all indicated ample safety factors; thus, the slide should not have occurred if the intact strengths of the clay shale foundation had been mobilized simultaneously along the entire long path.

Short path cases indicated lower safety factors than equivalent long path cases. Even these lower safety factors were adequate to preclude a slide at intact strength levels except for Cases 3 and 4, which are marginal. Note that Case 3 is for the observed slide elevation and indicates the minimum short path factor of safety.

While a short path slide was not observed, Case 3 is the critical surface - shear strength combination for short path cases. Given the very low calculated safety factor and the strain softening nature of the highly anisotropic, brittle Pepper Shale, one can rationalize that the operational shear strength was fully mobilized in much of the foundation. The "large" strains incurred at such low safety factors

surely "set the stage" for the ultimate slide development as a long path surface.

(2) Case 6.

(a) Case 6 short path assumed that the operational strength of both the embankment and foundation beneath it had been reduced to residual (steady state) levels by the time the embankment reached elevation 497. The calculated safety factor of 0.53 is implausibly low implying that a slide would have occurred long before the embankment reached elevation 497.

(b) Case 6 long path assumed that the operational strength of the embankment and the foundation beneath it had been reduced to residual levels, and that beyond the toe, the full intact strength could be mobilized. Such is strongly suggestive of a progressive slide initiating beneath a highly stressed (locally over-stressed) but marginally stable embankment/foundation (Case 3, short path) which develops into a long path slide having lateral extent similar to the observed slide (Case 6 long case). This case tacitly assumes that crossbed resistance, horizontal flexural stiffness or some other mechanism prevented development of a classic passive wedge breakout short of the long path slide observed.

(3) Case 7 demonstrates that assuming a uniformly low strength in the Pepper is not reasonable. Results for Case 7 short path indicate progressively lower safety factors with depth, implying

that sliding should have occurred at a location well below the observed slide elevation. Based on this, the assumption of uniformly low strength with depth can be discounted.

(4) Case 8 corresponds to Case 7, but is for long paths. Residual strength is assumed to exist beneath the dam, and intact strength from the toe to the downstream limit of the slide. All of the calculated safety factors are ample for stability. This indicates that sliding would not have occurred if the intact strength had been in effect on a portion of the slide surface.

b. <u>Cases examined - 1964 condition.</u>

(1) Cases 1, 2, 3, 4 and 5 are based on intact strength in the Pepper that varies with in-situ density, and therefore with depth. These cases were analyzed for long paths, assuming a passive wedge at the end of the berm. The calculated safety factors are high, but not realistic because the intact strength could not be in effect on the slide surface.

(2) Case 6 is for the sliding plane at elevation 370, and assumes residual strength within the limits of the actual slide plane, then intact shear strength from there downstream. The safety factor is 0.94 for the end of construction.

(3) Case 7 is the same as Case 6, but uses a higher shear strength in the embankment, assuming that grouting has restored the integrity of the embankment. The safety factor is 1.11.

(4) Case 8 uses a constant, low intact strength throughout the Pepper, for planes at elevations 340, 350, 360, 370, 380 and 390, for long paths. The safety factors decrease with lower elevation because of the increasing excess pore pressures, but are sufficient for stability.

(5) Case 9 is similar to Case 8, but uses a slightly higher, constant intact shear strength in the Pepper, so the safety factors are somewhat higher.

c. Cases examined - 1984 condition. Cases 1 through 9 represent the same combinations of assumptions as Cases 1 through 9 for 1964 conditions, but the excess pore pressures are lower due to drainage over the 20-year period. Safety factors computed are correspondingly higher.

d. Conclusions. The results of the new analyses, based on independent interpretations of shear strength and a thorough reassessment of excess pore pressures, tend to support the conclusions that were reached during the 1961-1964 analyses of the slide and redesign of the embankment. The conclusions are:

(1) If the Pepper clay shale stratum had been intact for the entire length of the slide plane, the slide would not have occurred. The slide occurred when the embankment and Pepper foundation are assumed to have reached a residual (steady state) strength condition within the limits of the embankment footprint and had intact strength for the remainder of the observed slide surface.

(2) Progressive failure in the Pepper Shale foundation best fits known conditions. It is known that a horizontal shear plane developed at elevation 370, about mid-depth of the Pepper Shale. The slide extended to a point 770 feet downstream of the axis of the embankment. Case 3 on Plate 64 for the short sliding plane, that is the plane emerging at the toe of the embankment, shows incipient sliding, using intact strength. Case 6, assuming a short sliding plane and residual strengths, produces a calculated factor of safety much less than unity, suggesting an unrealistic set of assumptions. Cases 1 through 5 for the long slide plane and intact shear strengths show adequate stability. However, when it is assumed that the slide has occurred in the short path and only residual strengths are available in that portion of the total path, then a factor of safety of only slightly less than 1.0 is derived as shown in the Long Slide Path, Case 6.

(3) The slide plane did not lie at the base of the Pepper as would have existed given uniformly low shear strength with depth.

(4) The full slide plane did not emerge at the toe of the dam, but came out about 770 feet downstream from the axis.

(5) There was evidence of progressive movement along the long slide surface expressed by the difference between the observed shear movements at the upstream and the downstream limits of the slide, and by the difference in times at which the displacements occurred.

(6) Cases 6-long path and 7-long path are most consistent with observed behavior and with previous analyses.

(7) Over the 20 years since completion of the dam, the calculated safety factor has increased 12 to 16 percent, due to dissipation of excess pore pressure.

8-04. Lessons Learned. The experiences gained from design, construction and operation of the Waco Dam Project have brought out several significant lessons, some of which are considered to be new contributions to the understanding of the behavior of clay shales.

a. Clay shales can develop high pore pressures due to the imposition of an embankment load. When Waco Dam was originally designed in the mid-1950's, conventional opinion was that clay shales could not develop positive pore pressures. This opinion was based on the belief that since they were highly over-consolidated they would exhibit dilative behavior. The only known previous case in which high pore pressures were measured in a clay shale foundation was at Fort Peck Dam, Montana, after the slide that occurred on 22 September 1938. However, the significance of the pore pressures was not recognized, either with respect to why they developed or what influence they had on stability. The high pressures measured were not used in the redesign of Fort Peck Dam, and there was no attempt to predict or consider excess pore pressures in the design of subsequent dams on clay shales until the slide at Waco Dam. After that event, other dams were instrumented and attempts were made to predict pore pressures in

design. A laboratory research program was begun by the Corps of Engineers, which ultimately led to the development, in 1982, of the ability to estimate pore pressures on the basis of the ratio of axial stiffness to radial stiffness, determined in a laboratory triaxial compression test (Leavell, et al., 1982).

b. The rate of dissipation of excess pore pressure is likely to be very slow because of the extremely low permeability of clay shales. Therefore, drainage and reduction of pore pressure during the usual few years of construction cannot be relied upon to provide a significant increase in calculated safety factors. Thus, using controlled rate of loading by limiting the amount of fill height that can be placed, for example on a weekly basis, may be a benefit by allowing time to evaluate the amount of excess pore pressure, but will do little to reduce it.

c. Clay shales are likely to have a very low residual (steady state) shear strength (angle of internal friction as low as 4 degrees) after a relatively small amount of deformation. This is caused by the brittleness of the material and by the type of clay minerals present. As a part of the testing associated with the analysis of the slide, specimens with a pre-cut shearing plane were used. A subsequent research program led to the development by the Corps of Engineers of a uniform test procedure for determination of residual (steady state) strength.

d. The Waco experience reiterated the significance of "minor" geologic details (Terzaghi, 1929) on the safety of dams. The extent

and nature of the slide were affected by minor geologic details, such as the location of cross-joints in the Pepper Formation and a horizontal bedding defect that had been weakened before the dam was built. Such features might not be specifically located in the course of the typical design investigations.

e. It is now considered necessary to have experienced geologists and/or engineers on the site during subsurface investigations to provide a competent and accurate interpretation of subsurface conditions.

f. It is essential to have communication and feed-back between design engineers and construction staffs. The construction personnel must understand the basis of design and of the construction plans and specifications. If differing or unexpected conditions are encountered, the designers should be involved in resolution of circumstances. Design personnel should make routine periodic visits to the construction site, especially when foundations are stripped and excavated.

g. The observational approach should be habitually used on dam projects to follow-up on design assumptions and to provide for changes if construction conditions do not satisfactorily conform with them. This principle was used to a great extent during the repair and completion of Waco Dam, and has been effectively used on subsequent dams on clay shales.

8-05. <u>Dam Safety</u>. The Fort Worth District has a strong commitment to dam safety. The Waco dam has undergone five inspections by teams

of geotechnical engineers and geologists as part of the program for Continued Evaluation of Completed Civil Works Projects. Instrumentation is being read and evaluated on a scheduled basis. However, much of the instrumentation is no longer functional. This is especially a problem in the slide area where high pore pressures still exist, and some creep movement is possibly occurring.

8-06. Recommendations. It is recommended that funds be made available to upgrade and replace instrumentation so that the embankment stability can continue to be adequately monitored. All observations and data that are available indicate that the embankment is functioning satisfactorily.

WACO DAM
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TABLE 1

EMBANKMENT QUANTITIES

Material	Volume cu yds
Impervious fill	8,513,900
Shale fill (from spillway excavation)	5,617,900
Random fill	1,536,300
Drainage blanket	284,300
Riprap	184,000
Bedding	72,600
Total	16,209,000

TABLE 2

SUMMARY OF DESIGN DATA

Feature	Moist Weight lb/cu ft	Type Test	Angle of Internal Friction Phi, degrees	Cohesion c ton/sq ft
Embankment				
Slide Area:				
Above El 450	130	Q	0	1.0
Below El 450	130	Q	0	0.4
Outside Slide Area:	130	Q	3	1.0
		R	12	0.4
		S	22	0.0
Foundation				
Overburden: (Right floodplain)	122	Q	0	1.0
Plane at El 388	123	Q	0	0.5
Plane at El 368	120	Q	0	0.7
		R	15	0.4
		S	25	0.0
Primary:	136	R	34	0.4
		S	34	0.4
Betonite seams:		Q	19	0.4
		S	19	0.4
Pepper Shale: (slide area)	130	S (residual)	8	0.0
Pepper Shale: (intact)			14	0.2
Pepper-Del Rio contact:		S	10-15	0.0
Del Rio Shale:	136	Q	28	0.6
		R	20	0.1
		S	19	0.4

TABLE 3

CONSTRUCTION CONTROL AND RECORD DATA

Feature	CONSTRUCTION CONTROL DATA (FIELD TESTS)		RECORD SAMPLE DATA (SWD LABORATORY TESTS)				No. of Tests
				SHEAR STRENGTH			
	Dry Weight lb/cu ft	Moisture Content %	Dry Weight lb/cu ft	Type Test	Angle of Internal Friction Phi, degrees	Cohesion (c) ton/sq ft	
Sta 59+00 - Sta 80+00							
Shale*	93.7 to 127.2	11.6 to 27.3	107.0 to 115.0 Avg = 110.2	Q	1.5 - 26.2 Avg = 9.8	0.9 - 2.4 Avg = 1.4	8
				R	8.3 - 18.9 Avg = 14.3	0.0 - 0.9 Avg = 0.5	9
				S	16.2 - 23.2 Avg = 19.2	0.0 - 0.3 Avg = 0.1	9
Sta 59+00 - Sta 180=45							
Impervious	96.3 to 128.8	10.0 to 26.0	104.0 to 121.0 Avg = 111.7	Q	0.0 - 33.1 Avg = 10.5	0.6 - 2.8 Avg = 1.4	33
				R	5.8 - 25.9 Avg = 14.4	0.2 - 2.1 Avg = 0.7	32
				S	15.7 - 31.8 Avg = 23.6	0.0 - 0.5 Avg = 0.2	32
Random	104.8 to 123.0	9.7 to 20.2					

*Shale material was not used in constructing the embankment between Sta 80+00 and Sta 180+45.

TABLE 3

CONSTRUCTION CONTROL AND RECORD DATA

Feature	CONSTRUCTION CONTROL DATA (FIELD TESTS)		RECORD SAMPLE DATA (SWD LABORATORY TESTS)				No. of Tests
				SHEAR STRENGTH			
	Dry Weight lb/cu ft	Moisture Content %	Dry Weight lb/cu ft	Type Test	Angle of Internal Friction Phi, degrees	Cohesion (c) ton/sq ft	
Sta 51+00 - Sta 59+00							
Impervious	103.9 to 118.5	13.1 to 19.4	97.0 to 128.8 Avg = 115.9	Q	0.0 - 25.2 Avg = 7.7	0.5 - 2.1 Avg = 1.2	81
				R	9.6 - 34.0 Avg = 18.3	0.0 - 1.2 Avg = 0.5	74
				S	16.3 - 36.9 Avg = 26.6	0.0 - 0.7 Avg = 0.3	72
Shale			106.0 to 110.0 Avg = 108.3	Q	4.7 - 12.0 Avg = 7.7	1.2 - 1.5 Avg = 1.3	4
				R	10.7 - 17.7 Avg = 15.0	0.3 - 0.5 Avg = 0.4	4
				S	17.7 - 19.8 Avg = 19.0	0.1 - 0.3 Avg = 0.2	4
Random	99.0 to 123.4	9.1 to 21.8					

TABLE 3

CONSTRUCTION CONTROL AND RECORD DATA

Feature	CONSTRUCTION CONTROL DATA (FIELD TESTS)		RECORD SAMPLE DATA (SWD LABORATORY TESTS)				No. of Tests
				SHEAR STRENGTH			
	Dry Weight lb/cu ft	Moisture Content %	Dry Weight lb/cu ft	Type Test	Angle of Internal Friction Phi, degrees	Cohesion (c) ton/sq ft	
<u>Sta 34+00 - Sta 51+00</u>							
Impervious	96.3 to 128.8	10.0 to 26.0	101.0 to 120.0 Avg = 112.6	Q	0.1 - 20.9 Avg = 8.0	0.7 - 2.8 Avg = 1.5	39
				R	4.5 - 25.2 Avg = 13.5	0.5 - 2.1 Avg = 0.8	31
				S	18.0 - 32.1 Avg = 24.1	0.0 - 0.4 Avg = 0.2	28
Shale	93.7 to 127.2	11.6 to 27.3	104.0 to 110.0 Avg = 107.5	Q	3.0 - 17.7 Avg = 6.1	1.0 - 2.4 Avg = 1.6	10
				R	8.0 - 15.4 Avg = 11.0	0.3 - 0.9 Avg = 0.6	7
				S	16.8 - 22.3 Avg = 18.6	0.0 - 0.4 Avg = 0.1	7
Random	104.8 to 123.0	9.7 to 20.2					

TABLE 3

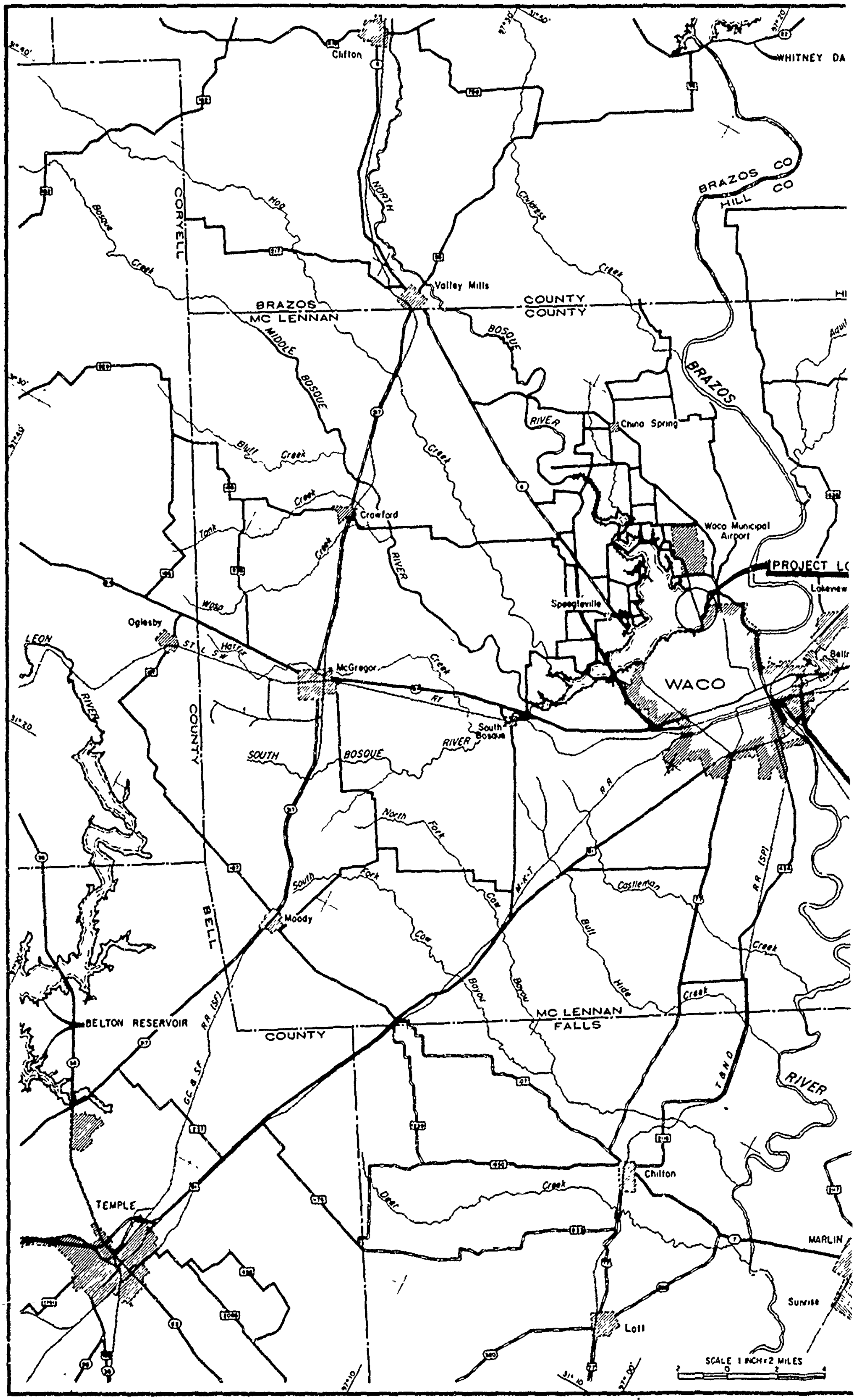
CONSTRUCTION CONTROL AND RECORD DATA

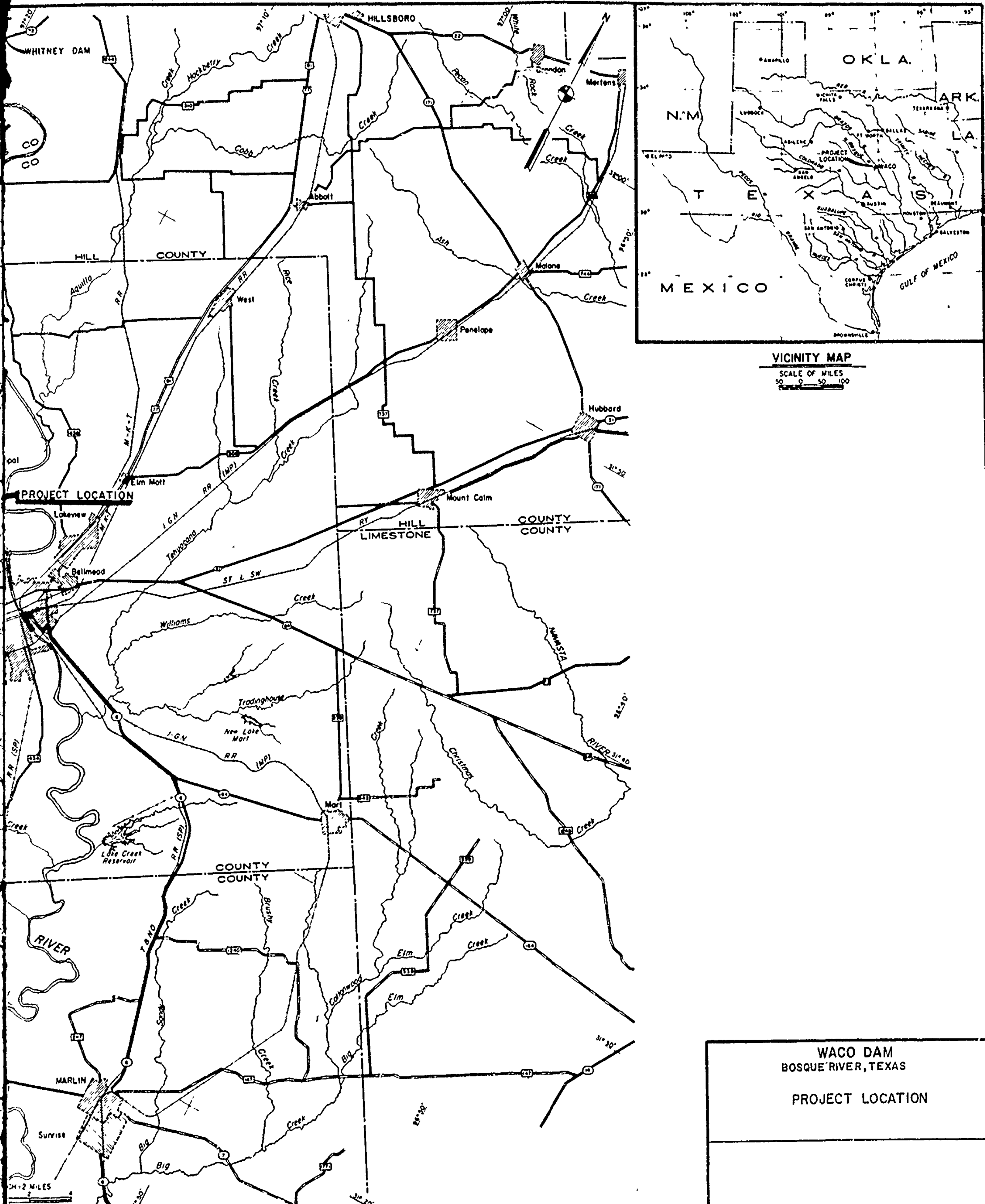
	CONSTRUCTION CONTROL DATA (FIELD TESTS)		RECORD SAMPLE DATA (SWD LABORATORY TESTS)				
				SHEAR STRENGTH			
Feature	Dry Weight lb/cu ft	Moisture Content %	Dry Weight lb/cu ft	Type Test	Angle of Internal Friction Phi, degrees	Cohesion (c) ton/sq ft	No. of Tests
Sta 0+00 - Sta 34+00							
Impervious	93.5 to 119.6	10.2 to 26.8	99.8 to 122.1 Avg = 110.3	Q	0.0 - 21.4 Avg = 6.0	0.6 - 2.4 Avg = 1.3	52
				R	13.6 - 15.0 Avg = 14.3	0.4 - 0.6 Avg = 0.5	2
				S	20.9 - 35.1 Avg = 27.3	0.1 - 0.4 Avg = 0.2	11
Shale	95.2 to 108.4	16.9 to 26.0	101.0 to 111.7 Avg = 105.3	Q	0.5 - 11.6 Avg = 4.1	0.7 - 2.3 Avg = 1.0	64
				R	13.8 - 18.6 Avg = 14.9	0.2 - 0.4 Avg = 0.3	5
				S	15.3 - 20.6 Avg = 18.3	0.2 - 0.5 Avg = 0.4	9
Random	100.7 to 120.3	11.6 to 20.4					

TABLE 4

SUMMARY OF SAFETY FACTORS

Location	Loading Condition	Safety Factor	
Slide area	Post-construction	1.15	Expected pore pressures
		1.11	Maximum pore pressures
Right floodplain - upstream	Post-construction	1.32	Overburden
Right floodplain - downstream	Post-construction	1.32	Overburden
Right floodplain - upstream	Post-construction	1.92	Upper Eagle Ford
Left floodplain - downstream	Post-construction	2.73	Lower Eagle Ford
River section - downstream	Steady seepage	1.58	Embankment
Left floodplain - downstream	Steady seepage	1.64	Lower Eagle Ford
Right floodplain - upstream	Rapid drawdown	1.37	Embankment
(Original upper slope	Rapid drawdown	1.29	Embankment)
River section - upstream	Critical pool	1.83	Embankment

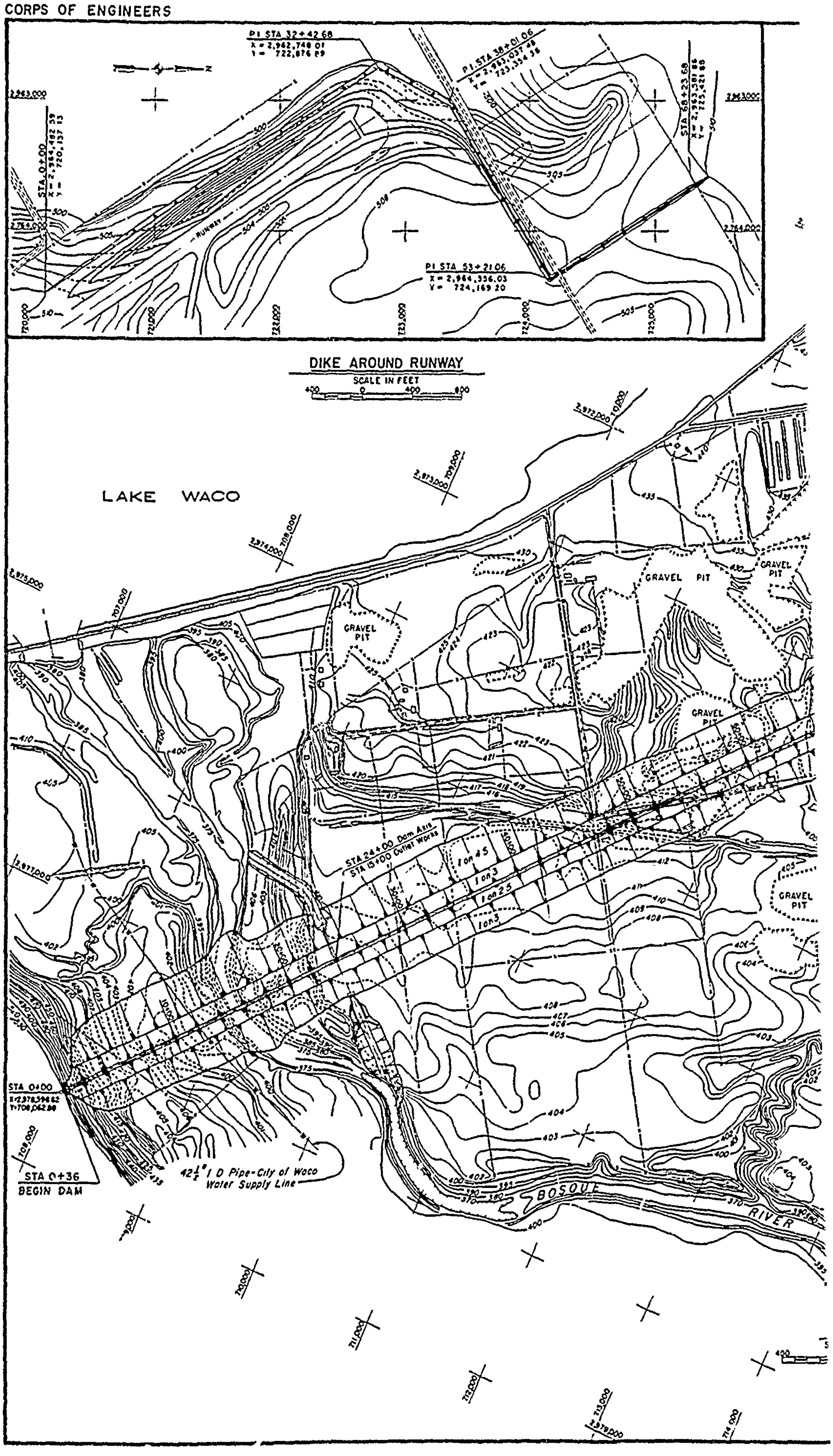

Clifton
WHITNEY DA
BRAZOS CO
HILL CO
CORYELL
Valley Mills
BRAZOS COUNTY
MC LENNAN COUNTY
BOSQUE RIVER
BRAZOS
MIDDLE BOSQUE
Chino Spring
Crawford
Waco Municipal Airport
PROJECT L
Lakeview
Speegleville
Oglesby
LEON RIVER
McGregor
WACO
South Bosque
SOUTH BOSQUE RIVER
BELL COUNTY
Moody
BELTON RESERVOIR
MC LENNAN FALLS COUNTY
RIVER
Chilton
TEMPLE
MARLIN
Sunrise
Lott
SCALE 1 INCH = 2 MILES

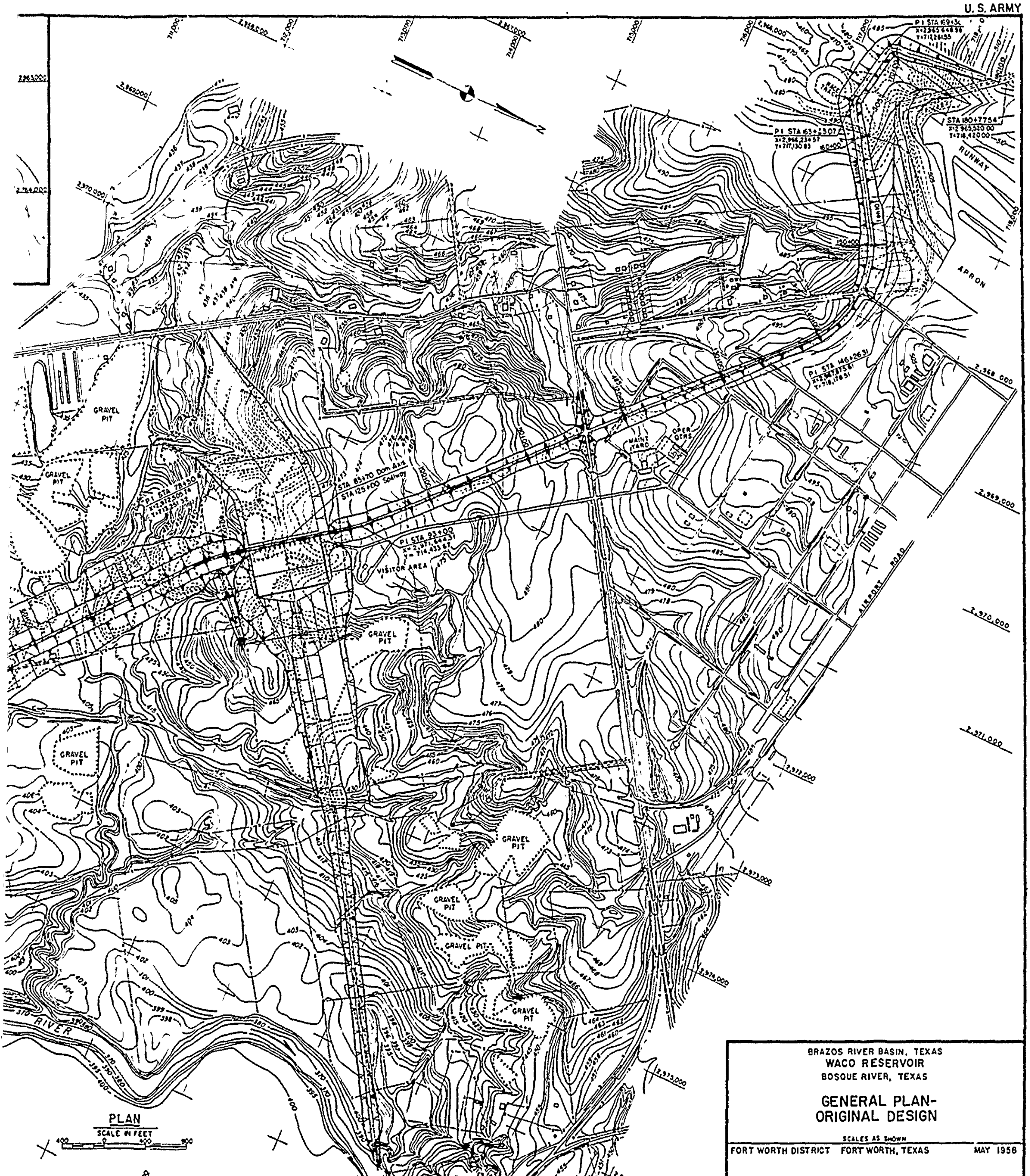


WACO DAM
BOSQUE RIVER, TEXAS

PROJECT LOCATION

PLATE I


PI STA 32+42.68
X = 2,962,748.01
Y = 722,876.89
PI STA 38+01.06
STA 0+00
X = 2,964,482.39
Y = 720,137.15
STA 66+23.68
RUNWAY
PI STA 53+21.06
X = 2,964,356.03
Y = 724,169.20
DIKE AROUND RUNWAY
SCALE IN FEET
400 0 400 800
LAKE WACO
GRAVEL PIT
GRAVEL PIT
GRAVEL PIT
GRAVEL PIT
GRAVEL PIT
STA 24+00 Dam Axis
STA 15+00 Outlet Works
1 on 4.5
1 on 3
1 on 2.5
1 on 3
STA 0+00
STA 0+36
BEGIN DAM
$42\frac{1}{2}$" I D Pipe-City of Waco
Water Supply Line
BOSQUE
RIVER

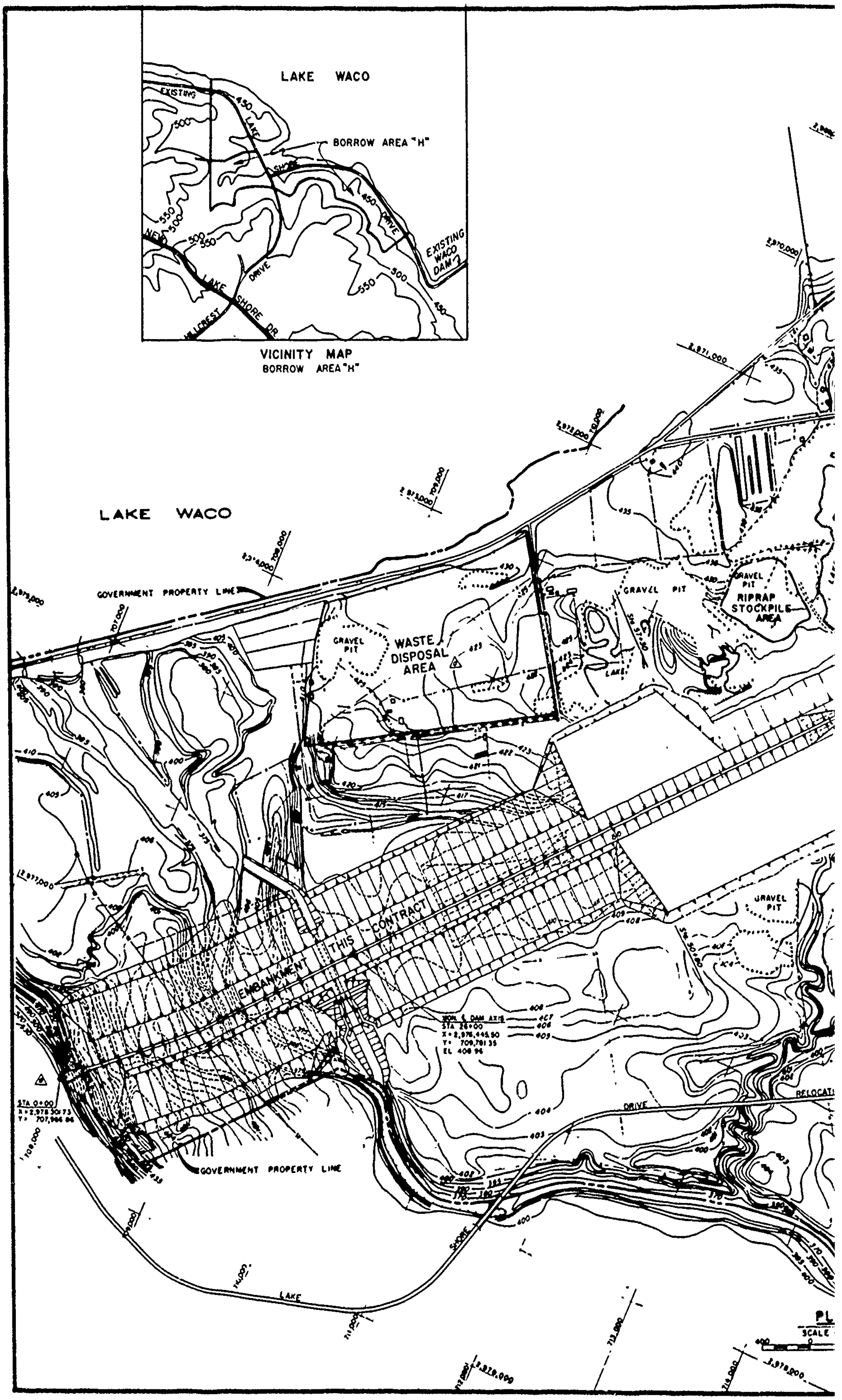


BRAZOS RIVER BASIN, TEXAS
WACO RESERVOIR
BOSQUE RIVER, TEXAS

GENERAL PLAN-
ORIGINAL DESIGN

SCALES AS SHOWN

FORT WORTH DISTRICT FORT WORTH, TEXAS MAY 1958



LAKE WACO
EXISTING
LAKE
SHORE
DRIVE
BORROW AREA "H"
EXISTING WACO DAM
NEW
HILLCREST
LAKE SHORE DR
VICINITY MAP
BORROW AREA "H"
LAKE WACO
GOVERNMENT PROPERTY LINE
GRAVEL PIT
WASTE DISPOSAL AREA
GRAVEL PIT
RIPRAP STOCKPILE AREA
EMBANKMENT THIS CONTRACT
GRAVEL PIT
GOVERNMENT PROPERTY LINE
DRIVE
SHORE
LAKE

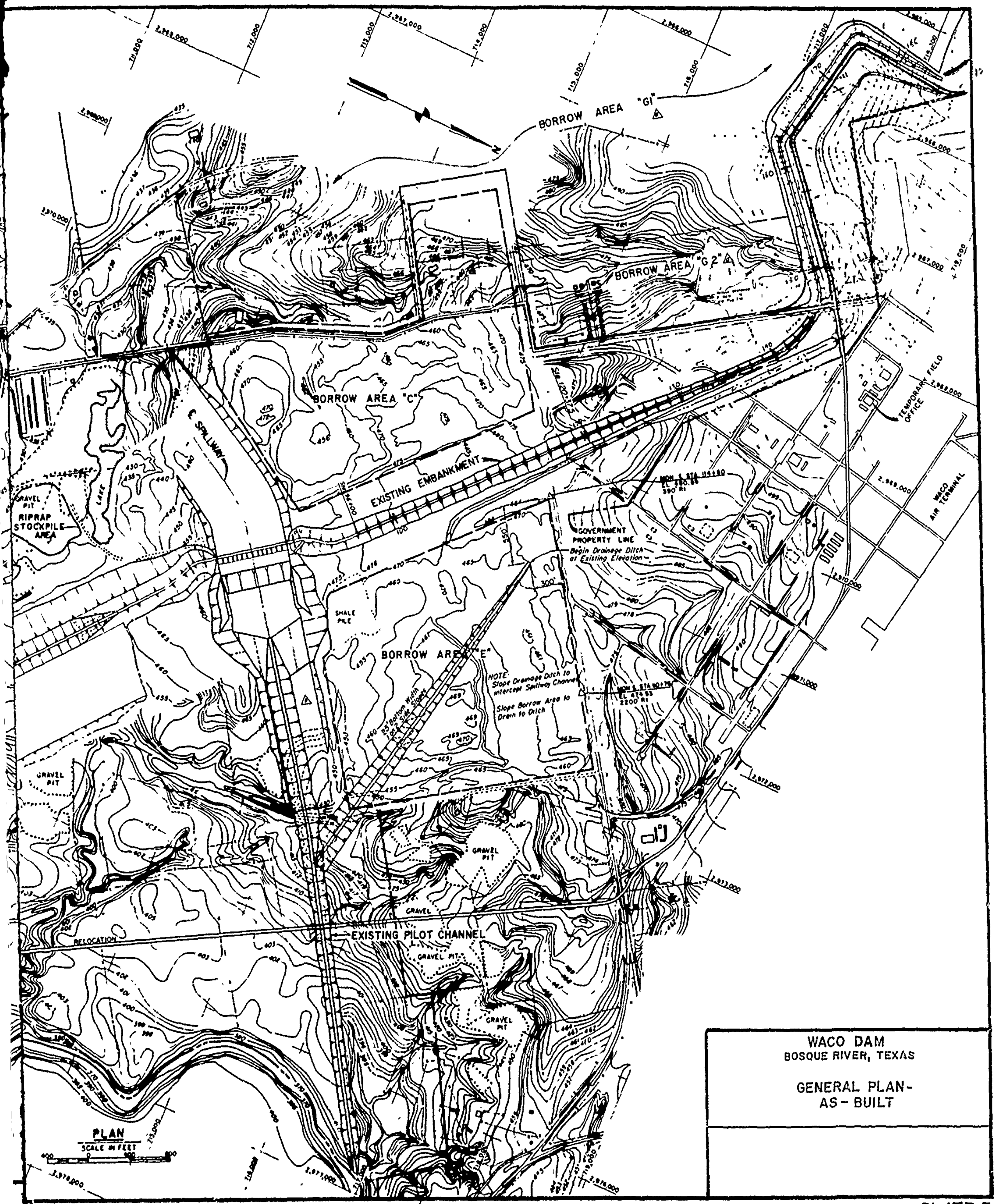
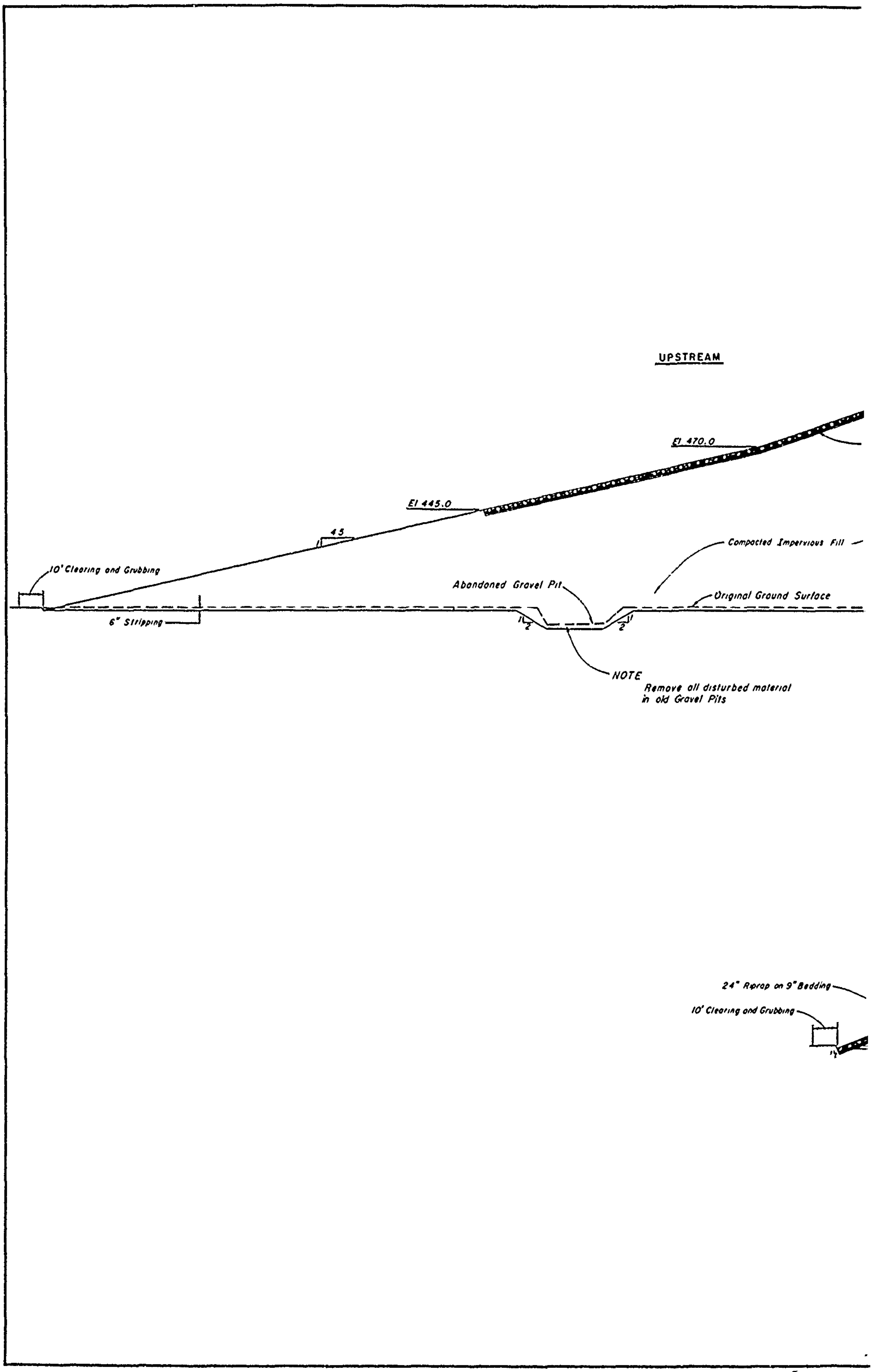
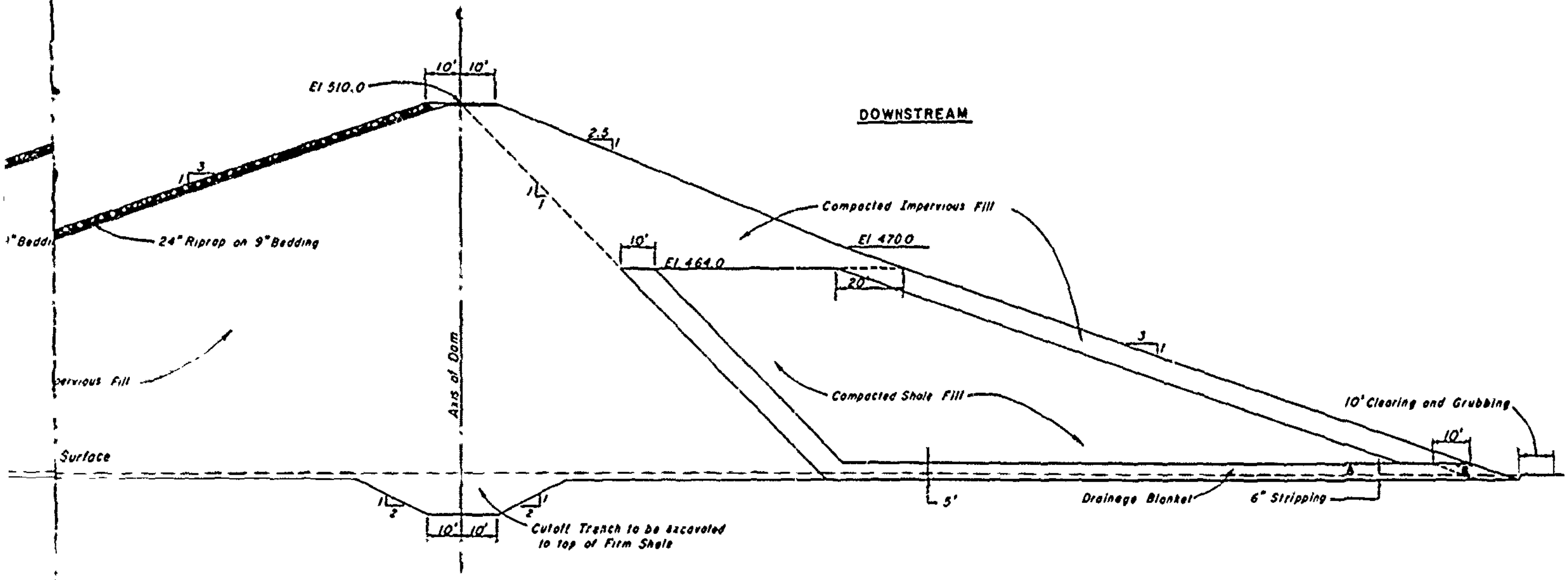

BORROW AREA "G1"
BORROW AREA "G2"
BORROW AREA "C"
BORROW AREA "E"
EXISTING EMBANKMENT
SPILLWAY
GRAVEL PIT
RIPRAP STOCKPILE AREA
SHALE PILE
GOVERNMENT PROPERTY LINE
Begin Drainage Ditch at Existing Elevation
NOTE: Slope Drainage Ditch to intercept Spillway Channel
Slope Borrow Area to Drain to Ditch
TEMPORARY FIELD OFFICE
WACO AIR TERMINAL
RELOCATION
EXISTING PILOT CHANNEL
PLAN
SCALE IN FEET
WACO DAM
BOSQUE RIVER, TEXAS
GENERAL PLAN-
AS-BUILT

PLATE 3



UPSTREAM
El 470.0
El 445.0
4.5
1
Compacted Impervious Fill
10' Clearing and Grubbing
Abandoned Gravel Pit
Original Ground Surface
6" Stripping
1
2
1
2
NOTE
Remove all disturbed material in old Gravel Pits
24" Riprap on 9" Bedding
10' Clearing and Grubbing

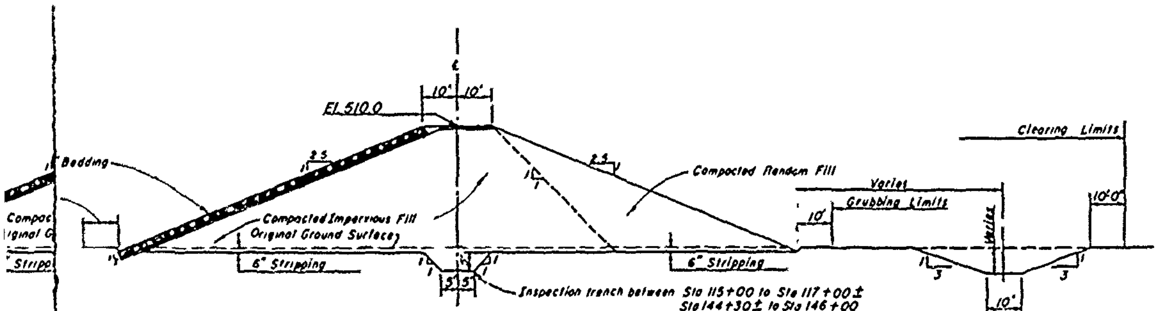


TYPICAL EMBANKMENT SECTION

STA. 34 + 12 TO STA. 79 + 90 ±

SCALE 1 INCH = 20 FEET

NOTE.
No drainage blanket will be placed above El. 464.0



TYPICAL EMBANKMENT SECTION

STA. 92 + 30 ± TO STA. 120 + 37.5 ±
STA. 143 + 00 ± TO STA. 144 + 60 ±

SCALE 1 INCH = 20 FEET

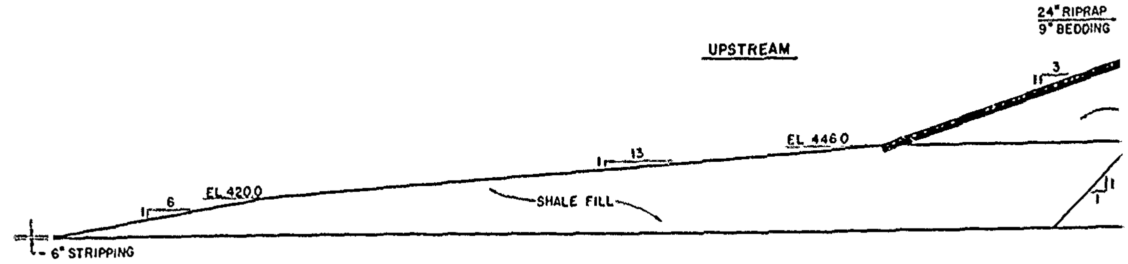
20 0 20 40

WACO DAM
BOSQUE RIVER, TEXAS

TYPICAL EMBANKMENT SECTION
ORIGINAL DESIGN

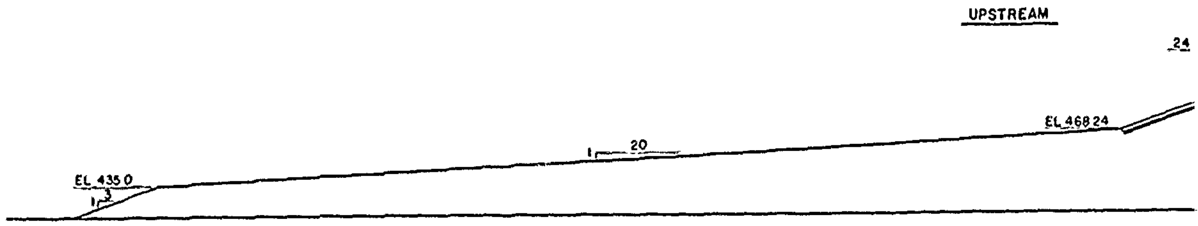
1 2 3 4 5

G



F

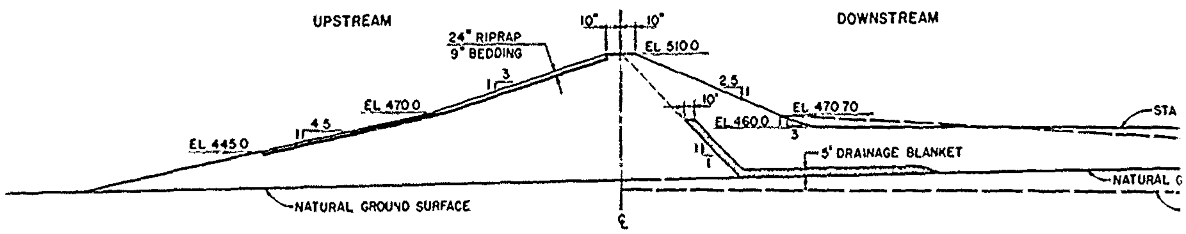
TY



E

TYP

D



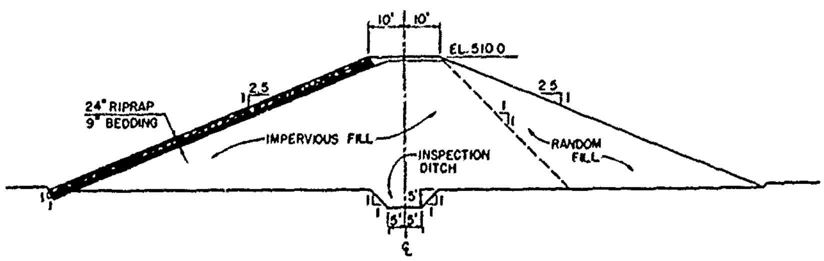
C

TYPICAL EMBANKMENT SECTION

STATION 61+00 TO 79+00

50 0 50 100
SCALE IN FEET

B



TYPICAL EMBANKMENT SECTION

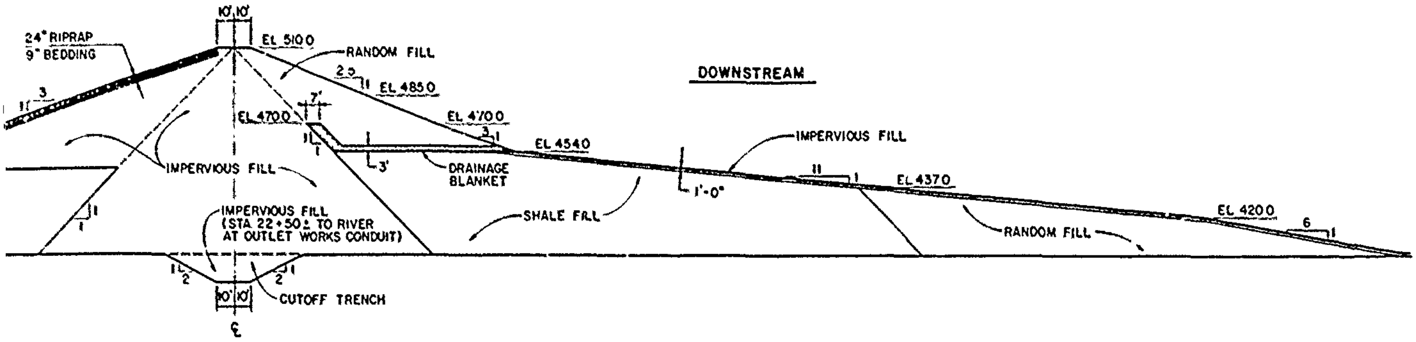
STATION 90+00 TO 180+45

10 0 10 20
SCALE IN FEET

A

1 2 3 4 5

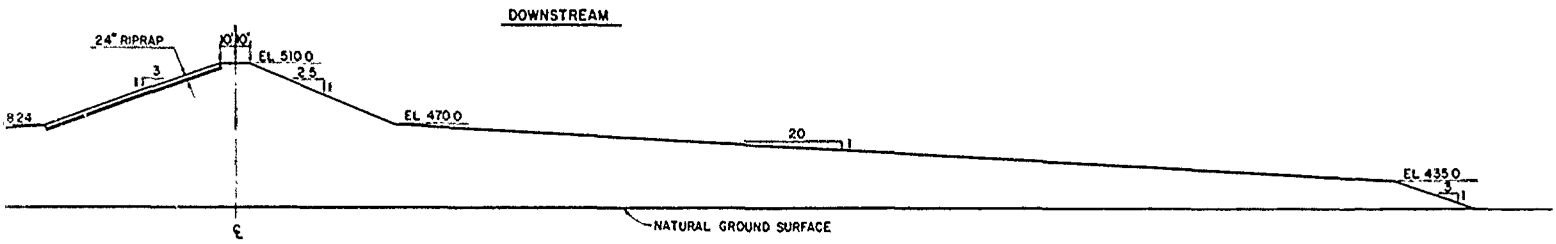
5 6 7 8 9 10



TYPICAL EMBANKMENT SECTION

STATION 0+00 TO 45+00

40 0 40 80

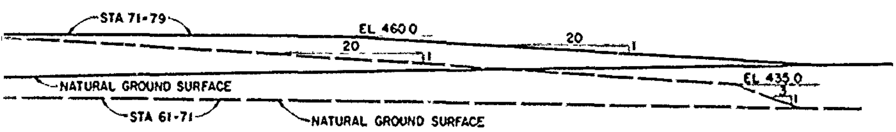


TYPICAL EMBANKMENT SECTION

STATION 50+00 TO 57+00

50 0 50 100

SCALE OF FEET



SYMBOL	DESCRIPTION OF REVISION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

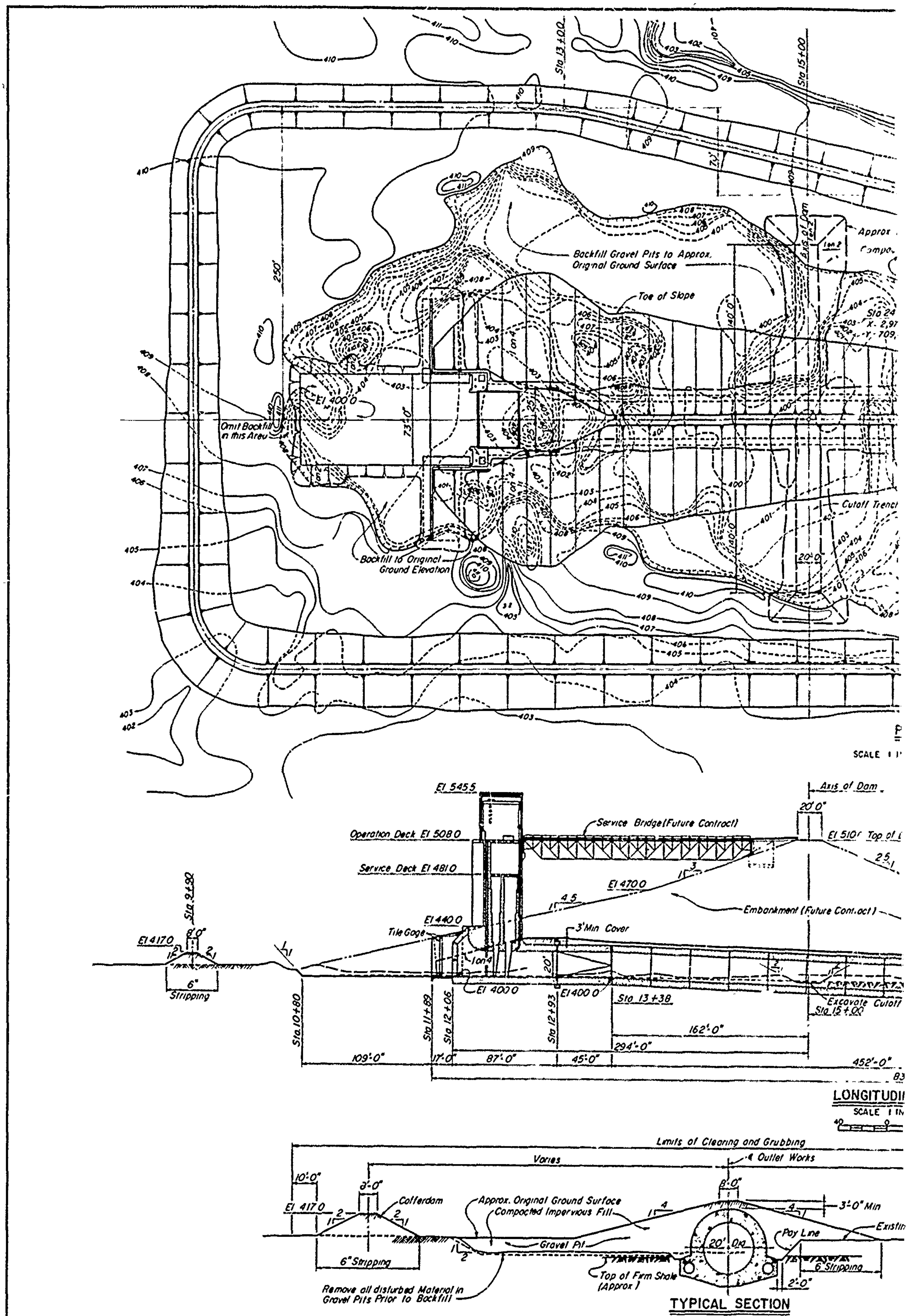
DESIGNED BY: ---------
DRAWN BY: ---------
CHECKED BY: ---------

BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

TYPICAL EMBANKMENT SECTIONS
AS-BUILT

SUBMITTED BY:
ENGINEER:
INV. NO.
DATED:
DRAWING NUMBER
SHEET NO.
of
SEQUENCE NO.

5 6 7 8



TYPICAL SECTION
SCALE 1 INCH = 20 FEET

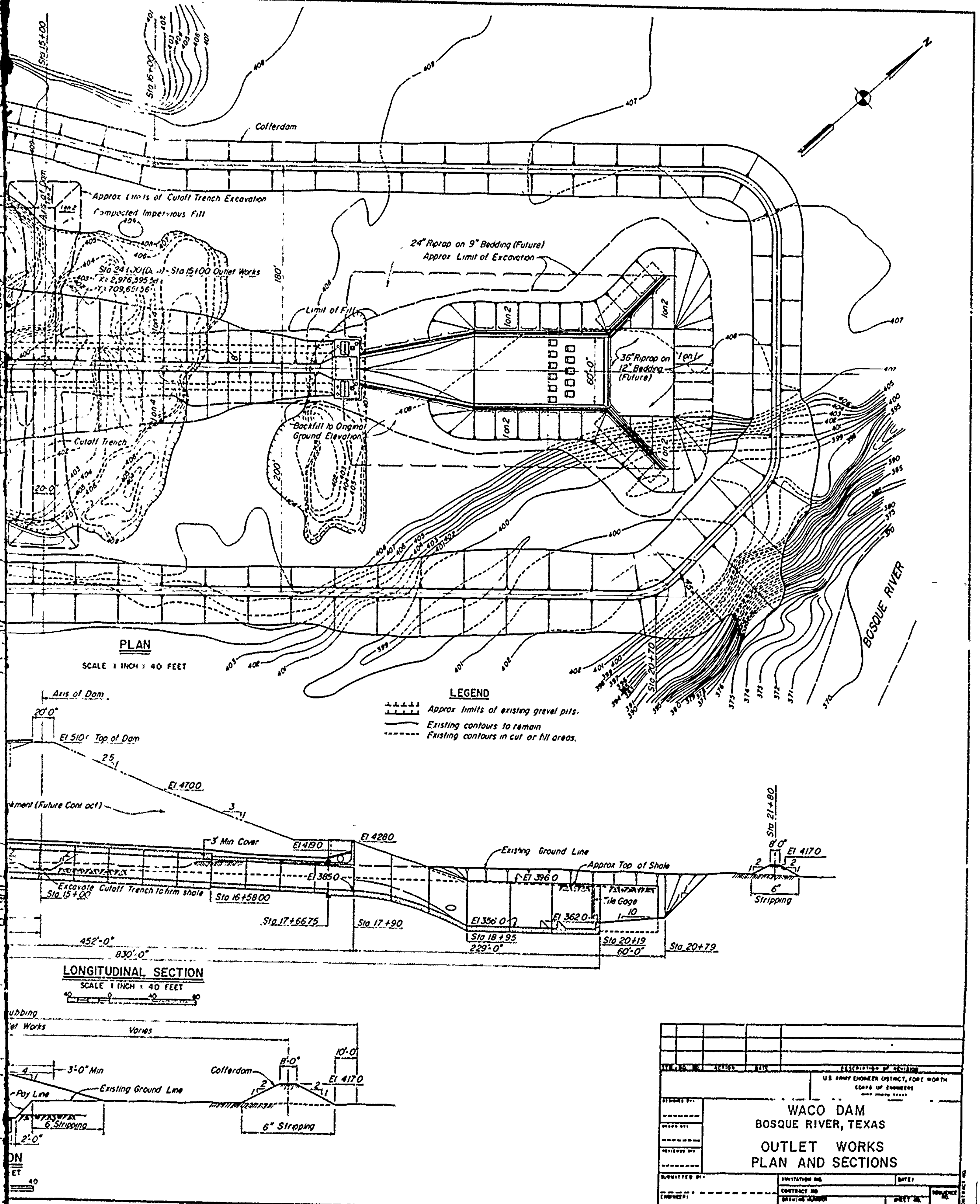
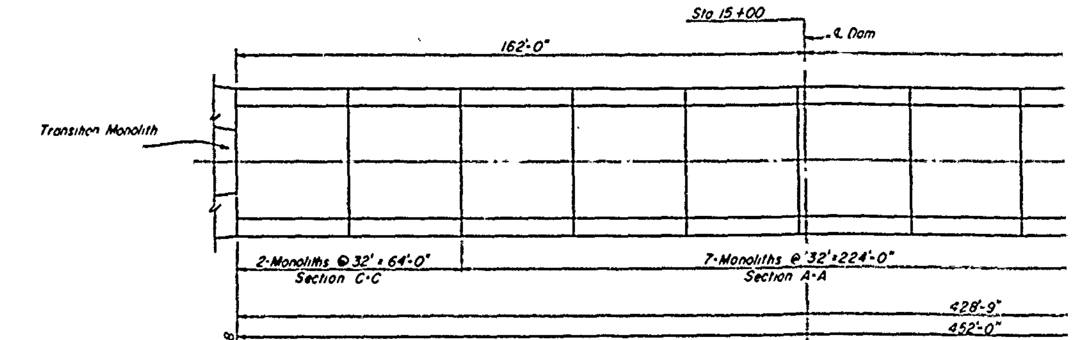
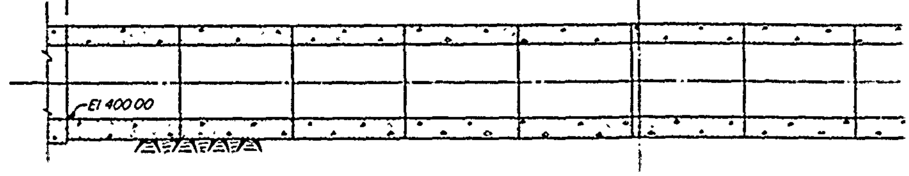

PLAN
SCALE 1 INCH = 40 FEET
LEGEND
Approx limits of existing gravel pits.
Existing contours to remain
Existing contours in cut or fill areas.
LONGITUDINAL SECTION
SCALE 1 INCH = 40 FEET
BOSQUE RIVER
Cofferdam
Approx Limits of Cutoff Trench Excavation
Compacted Impervious Fill
24" Riprap on 9" Bedding (Future)
Approx Limit of Excavation
Limit of Fill
36" Riprap on 12" Bedding (Future)
Backfill to Original Ground Elevation
Cutoff Trench
Axis of Dam
El 510.0 Top of Dam
El 470.0
El 419.0
El 428.0
Existing Ground Line
Approx Top of Shale
El 385.0
El 396.0
El 356.0
El 362.0
El 417.0
Excavate Cutoff Trench to firm shale
Sta 15+00
Sta 16+58.00
Sta 17+66.75
Sta 17+90
Sta 18+95
Sta 20+19
Sta 20+79
Sta 21+80
452'-0"
830'-0"
229'-0"
60'-0"
Stripping
6" Stripping
Existing Ground Line
Pay Line
Varies
3'-0" Min
U.S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
WACO DAM
BOSQUE RIVER, TEXAS
OUTLET WORKS
PLAN AND SECTIONS

PLATE 6



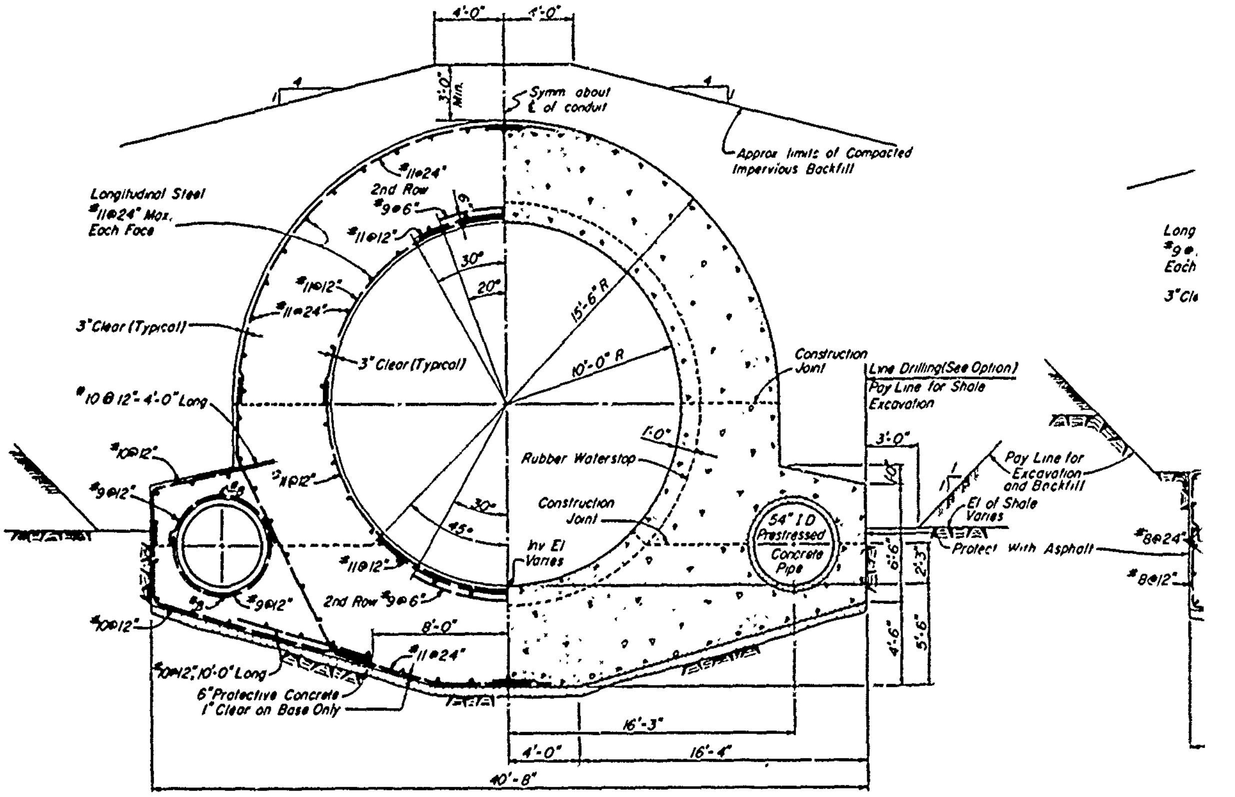
PLAN

SCALE 1 INCH = 20



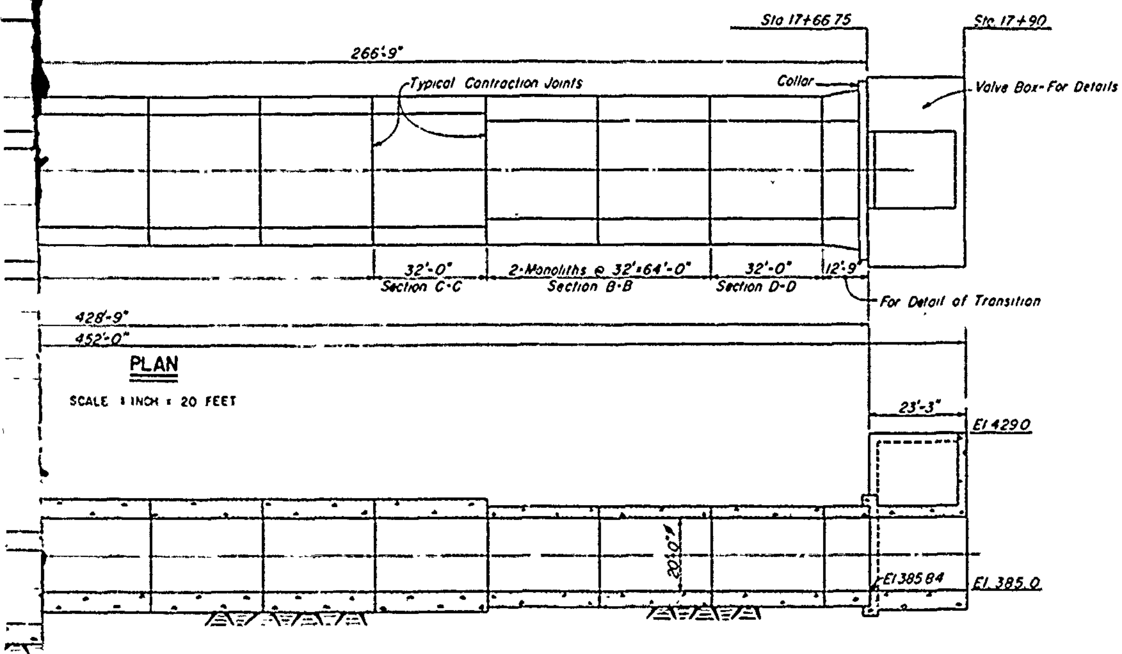
LONGITUDINAL S

SCALE 1 INCH = 20



SECTION A-A

SCALE: 1/4 INCH = 1 FOOT

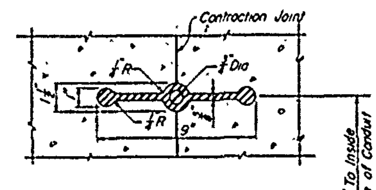


PLAN

SCALE 1 INCH = 20 FEET

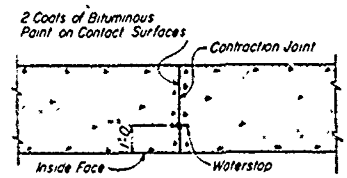
LONGITUDINAL SECTION

SCALE 1 INCH = 20 FEET



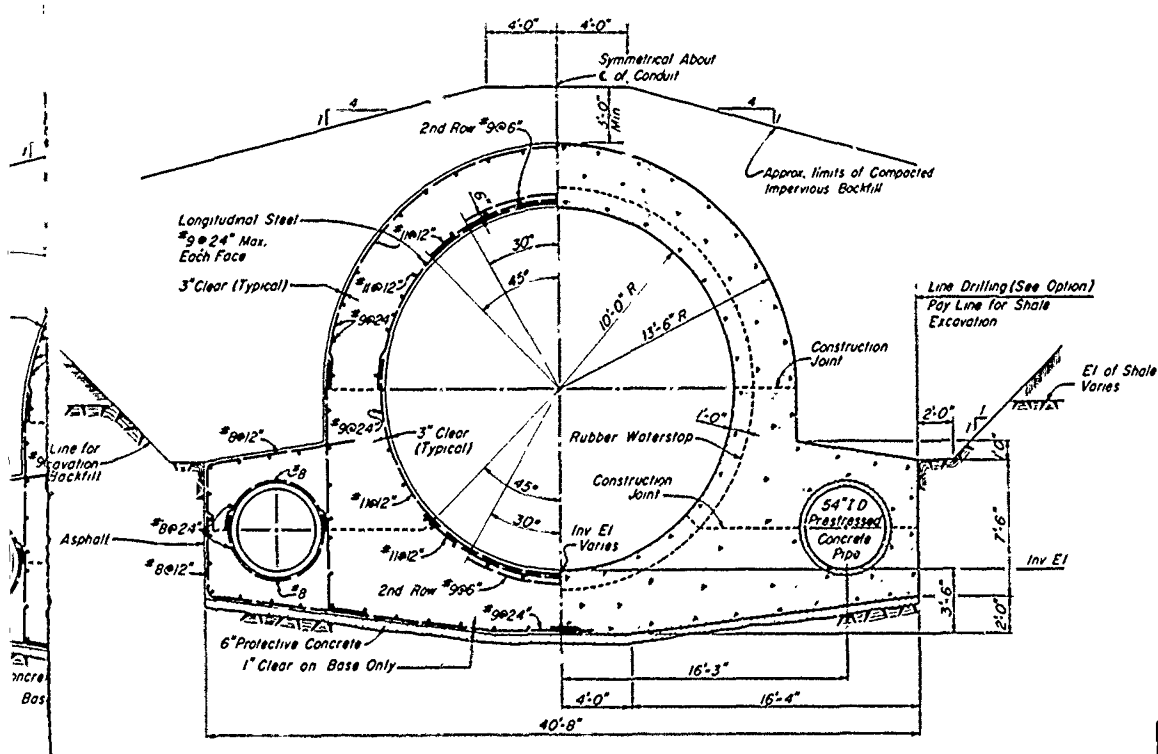
RUBBER WATERSTOP DETAIL

SCALE: 3 INCHES = 1 FOOT



TYPICAL CONTRACTION JOINT

SCALE 3/4 INCH = 1 FOOT



SECTION B-B

SCALE 1/2 INCH = 1 FOOT

U.S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

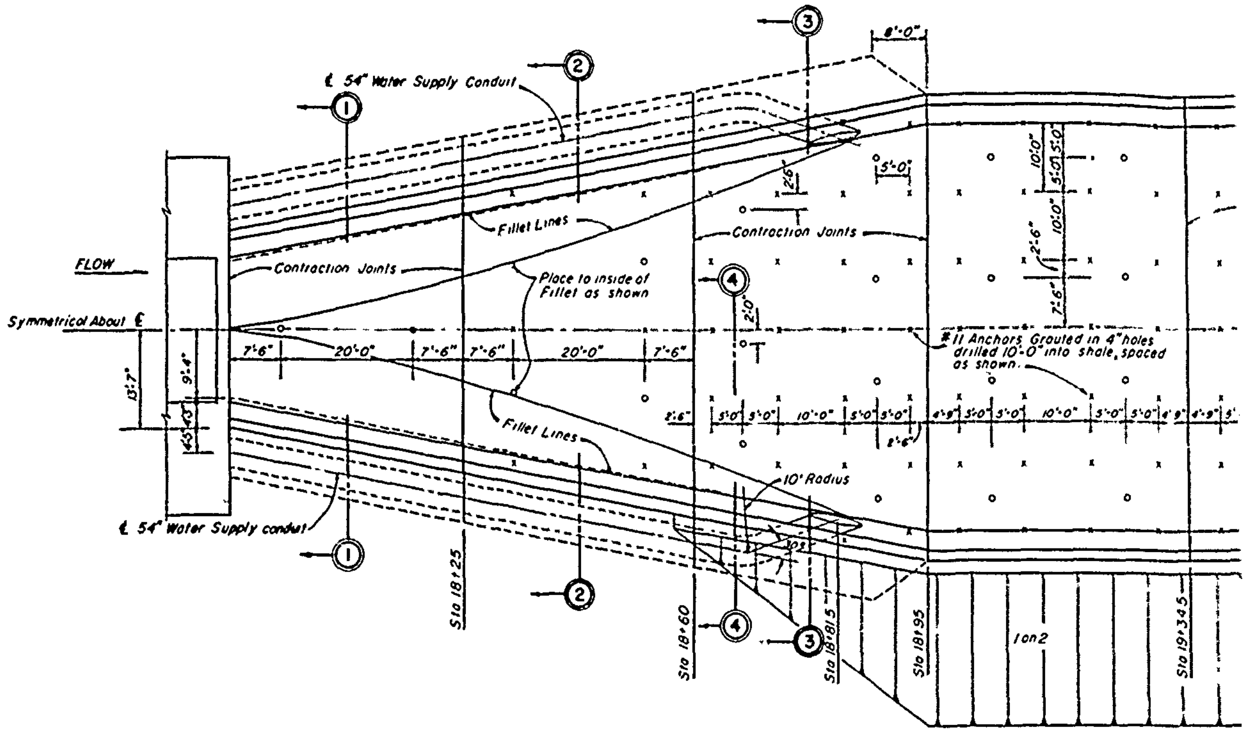
BRAZOS RIVER AND TRIBUTARIES, TEXAS
BOSQUE RIVER, TEXAS

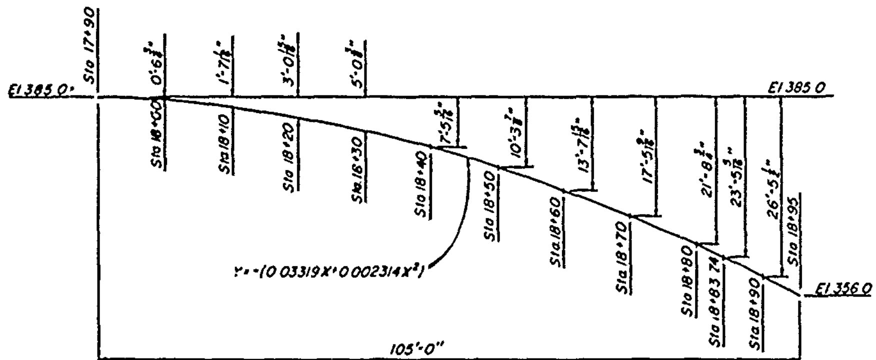
WACO DAM

OUTLET WORKS

CONDUIT - PLAN AND SECTION

PLATE 7

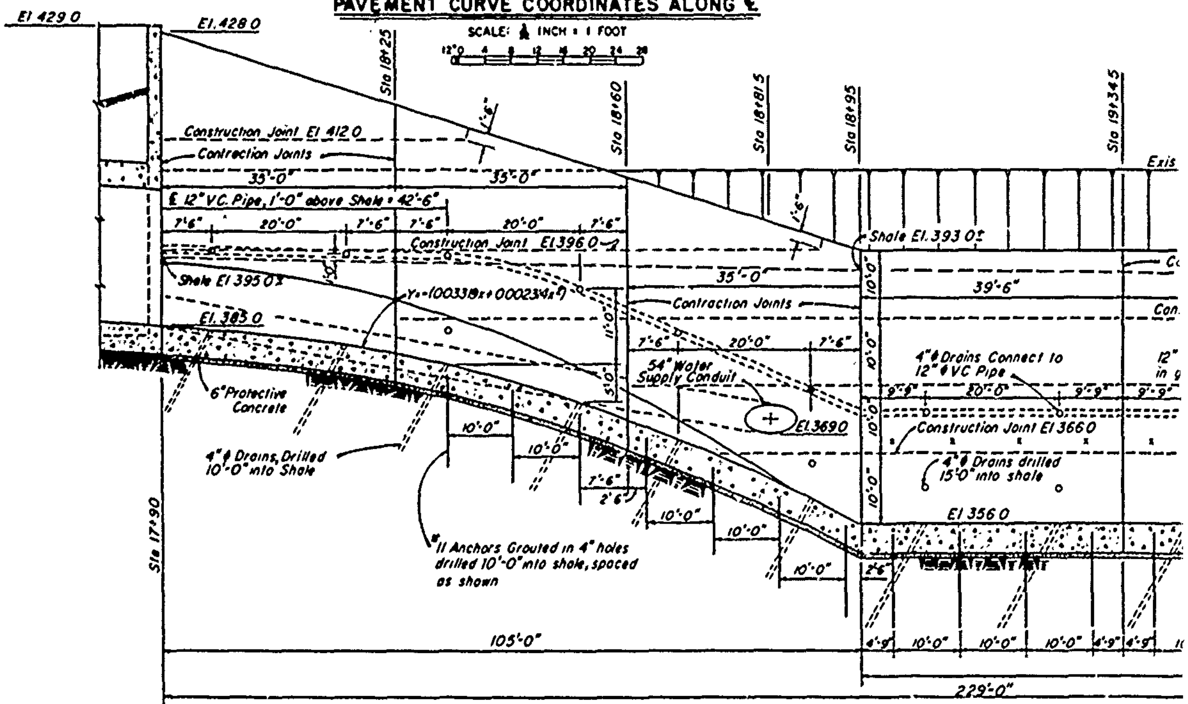




PAVEMENT CURVE COORDINATES ALONG ℄

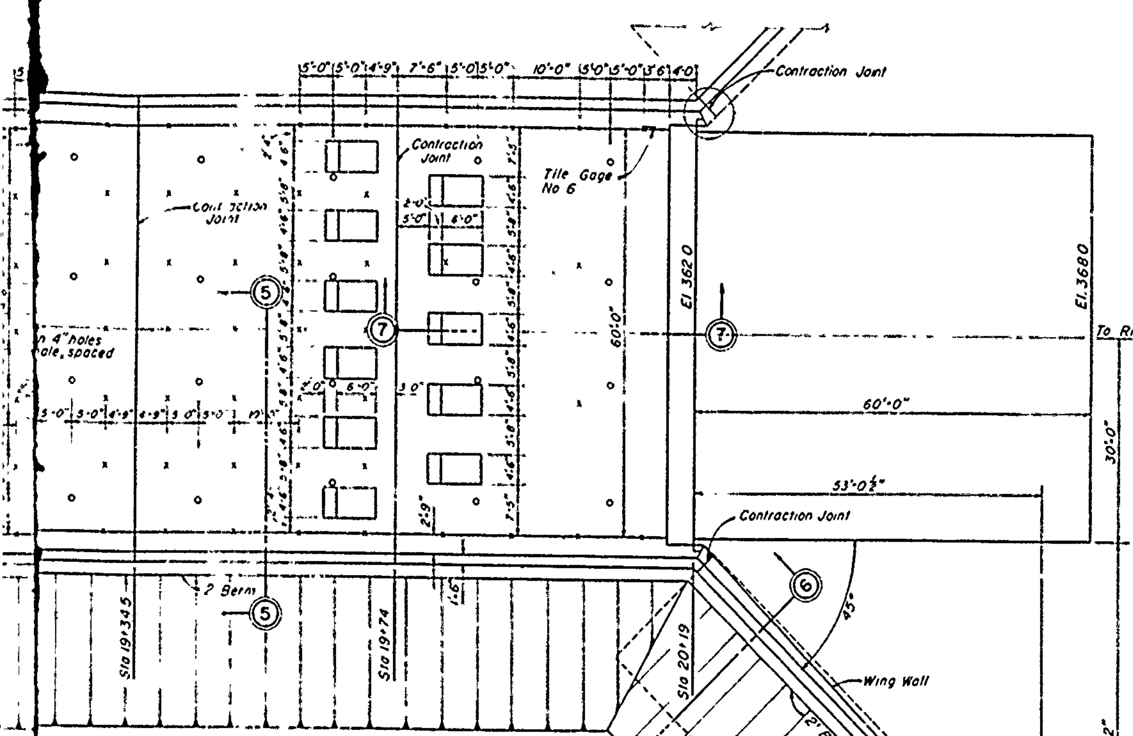
SCALE: 1/8 INCH = 1 FOOT

RADII TRANSITION		
STATION	R	A
17+90	10'-0"	7[illegible]"
18+00	9'-10[illegible]"	7[illegible]"
18+10	9'-6[illegible]"	7[illegible]"
18+20	8'-11[illegible]"	6[illegible]"
18+30	8'-2[illegible]"	6[illegible]"
18+40	7'-1[illegible]"	5[illegible]"
18+50	5'-10[illegible]"	4[illegible]"
18+60	4'-5[illegible]"	3[illegible]"
18+70	2'-8[illegible]"	2[illegible]"
18+80	0'-9[illegible]"	0[illegible]"
18+83.74	0'-0"	0"



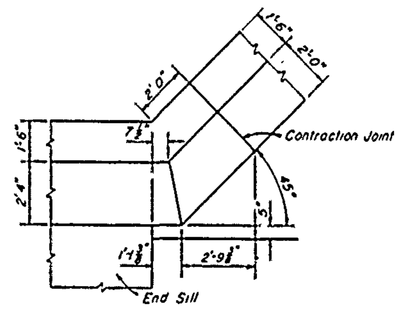
SECTION - STILLING BASIN

SCALE 1/8 INCH = 1 FOOT



PLAN

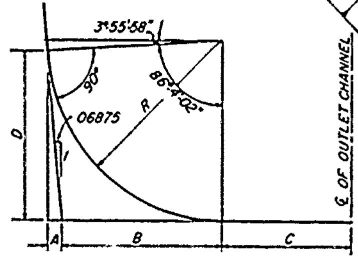
SCALE $\frac{1}{16}$ INCH = 1 FOOT



DETAIL "A"

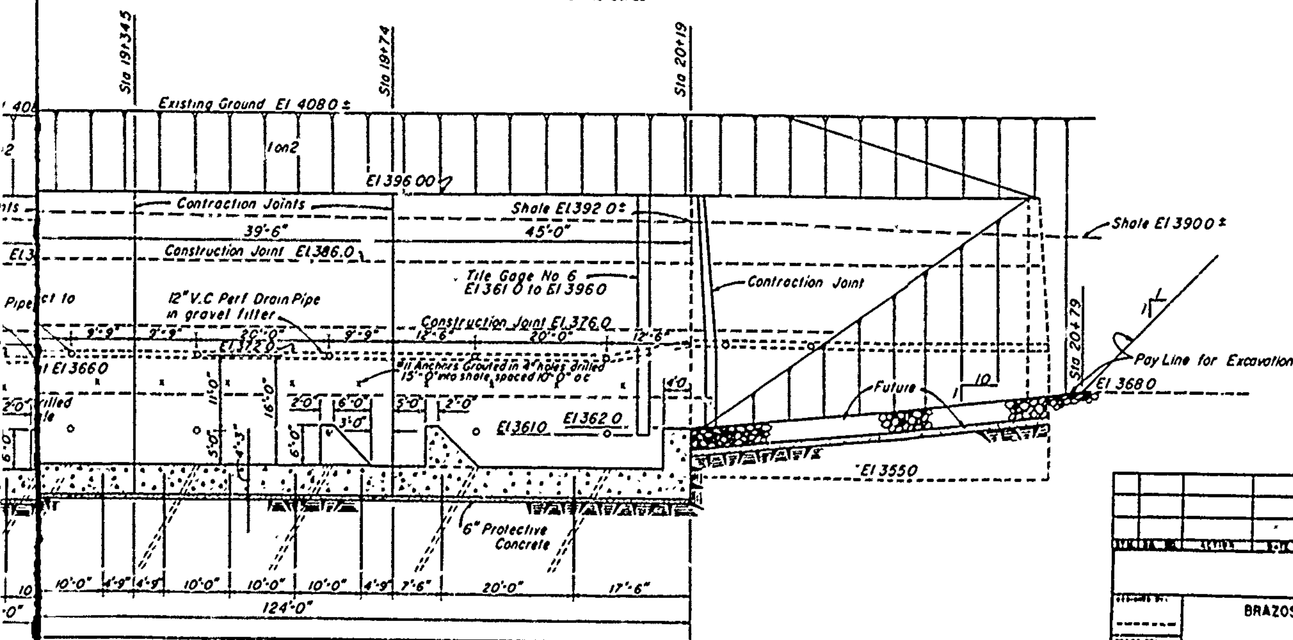
SCALE: $\frac{1}{2}$ INCH = 1 FOOT

RADII TRANSITION FROM 10'-0" TO 0						
[illegible]	ON	R	A	B	C	D
0	0	10'-0"	7[illegible]"	9'-4"	0	9'-3[illegible]"
5[illegible]"	0	9'-10[illegible]"	7[illegible]"	9'-2[illegible]"	1'-5[illegible]"	9'-2[illegible]"
9"	0	9'-6[illegible]"	7[illegible]"	8'-10[illegible]"	3'-9"	8'-10[illegible]"
3[illegible]"	0	8'-11[illegible]"	6[illegible]"	8'-4[illegible]"	6'-3[illegible]"	8'-4[illegible]"
0[illegible]"	0	8'-2[illegible]"	6[illegible]"	7'-7[illegible]"	9'-0[illegible]"	7'-7[illegible]"
0[illegible]"	0	7'-1[illegible]"	5[illegible]"	6'-8[illegible]"	12'-0[illegible]"	6'-7[illegible]"
3[illegible]"	0	5'-10[illegible]"	4[illegible]"	5'-6[illegible]"	15'-3[illegible]"	5'-6"
8[illegible]"	0	4'-5[illegible]"	3[illegible]"	4'-1[illegible]"	18'-8[illegible]"	4'-1[illegible]"
4[illegible]"	0	2'-8[illegible]"	2[illegible]"	2'-6[illegible]"	22'-4[illegible]"	2'-6[illegible]"
3[illegible]"	0	0'-9[illegible]"	0[illegible]"	0'-8[illegible]"	26'-3[illegible]"	0'-8[illegible]"
9[illegible]"	574	0'-0"	0"	0'-0"	27'-9[illegible]"	0'-0



SECTION-RADII TRANSITION

NOT TO SCALE



NG BASIN

1 FOOT

REVISION NO.	SYMBOL	DATE	DESCRIPTION OF REVISION	

U.S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

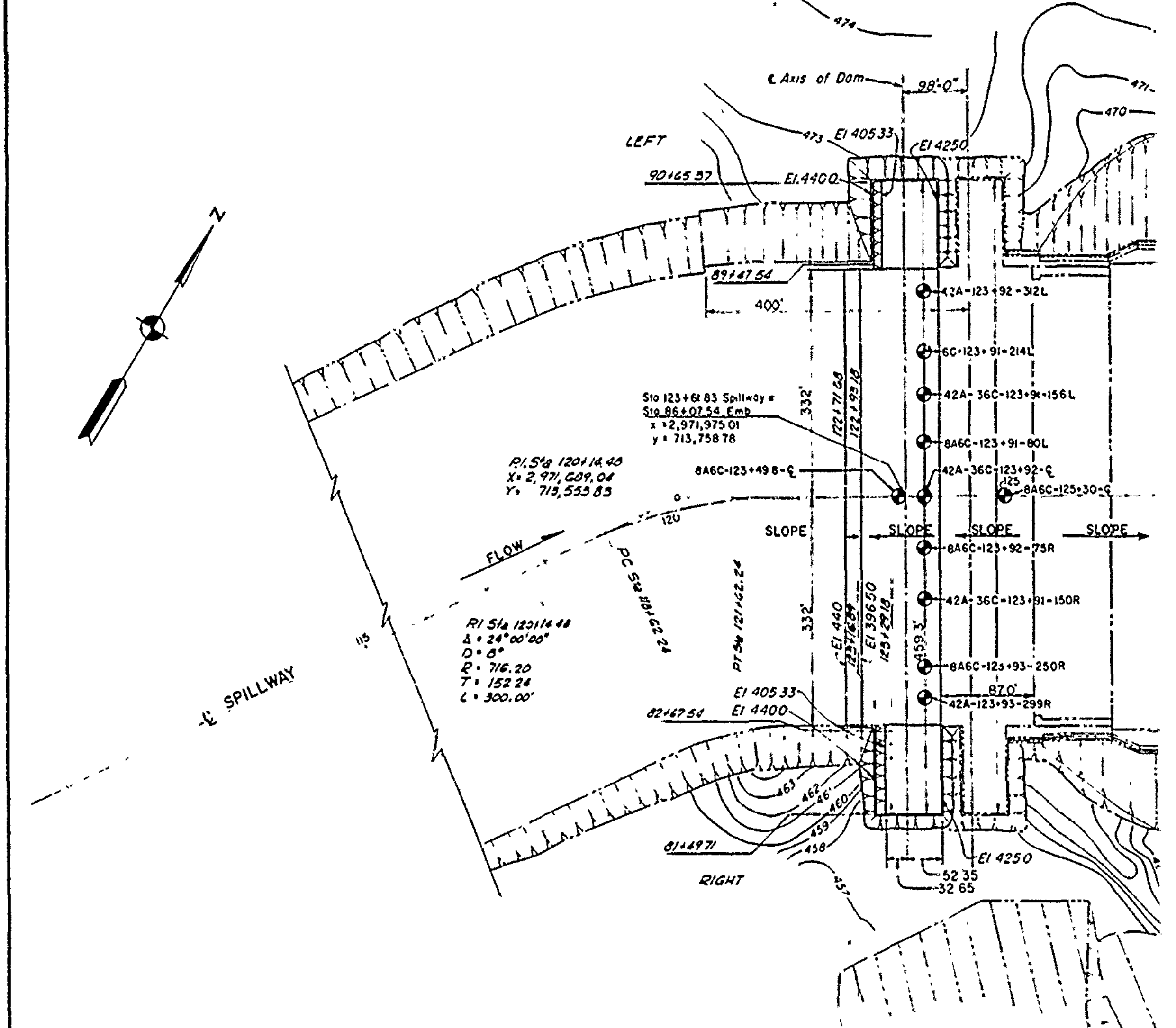
BRAZOS RIVER AND TRIBUTARIES, TEXAS
BOSQUE RIVER, TEXAS

WACO DAM

OUTLET WORKS

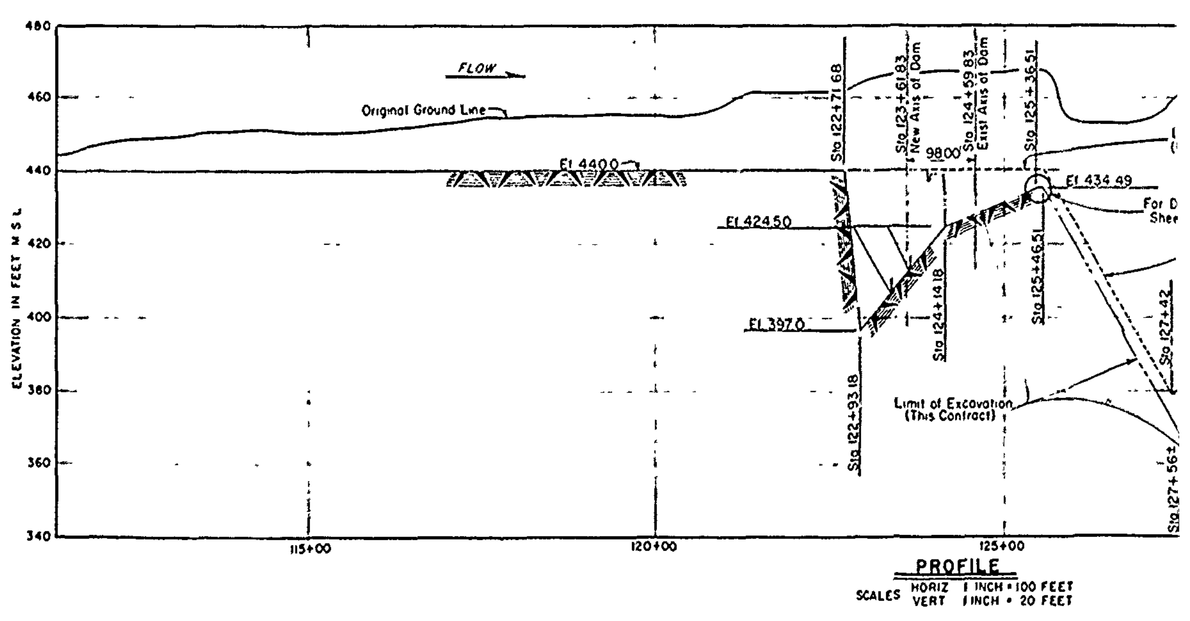
STILLING BASIN-PLAN AND SECTION

PLATE 8



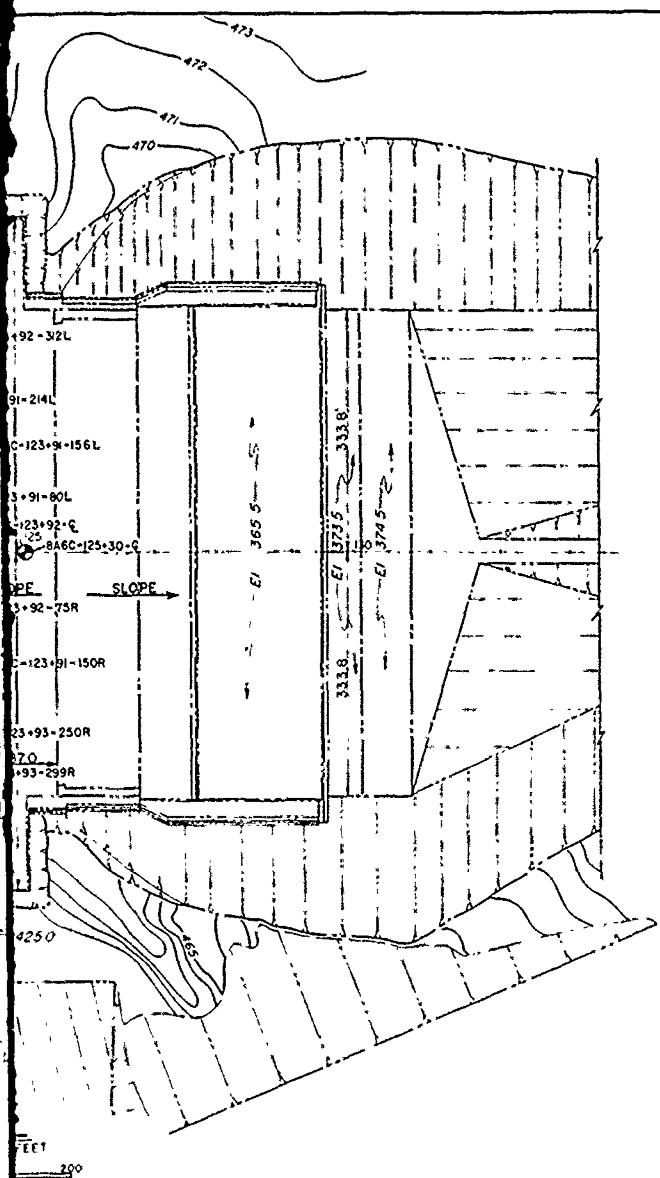
PLAN

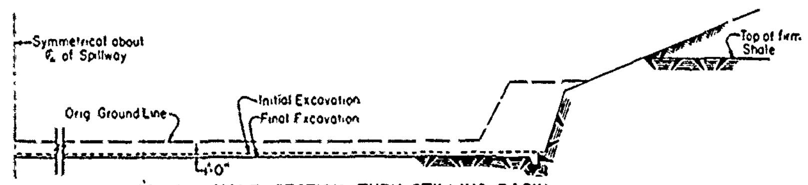
SCALE 1 INCH = 100 FEET



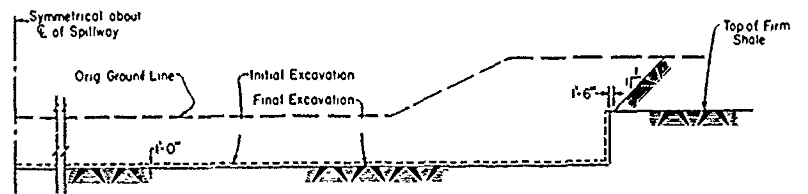
PROFILE

SCALES HORIZ 1 INCH = 100 FEET
VERT 1 INCH = 20 FEET

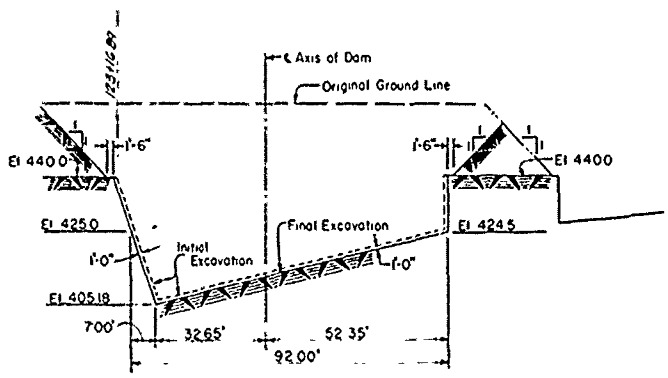




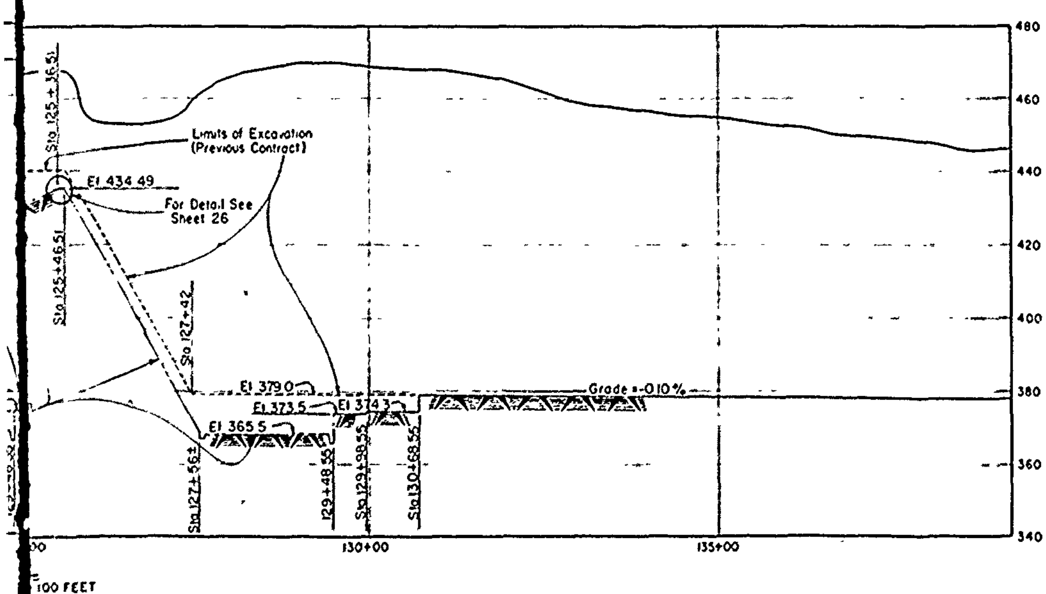
TYPICAL HALF SECTION THRU STILLING BASIN
SCALE 1 INCH = 40 FEET



TYPICAL HALF SECTION THRU WEIR AND NON-OVERFLOW SECTION
SCALE 1 INCH = 40 FEET

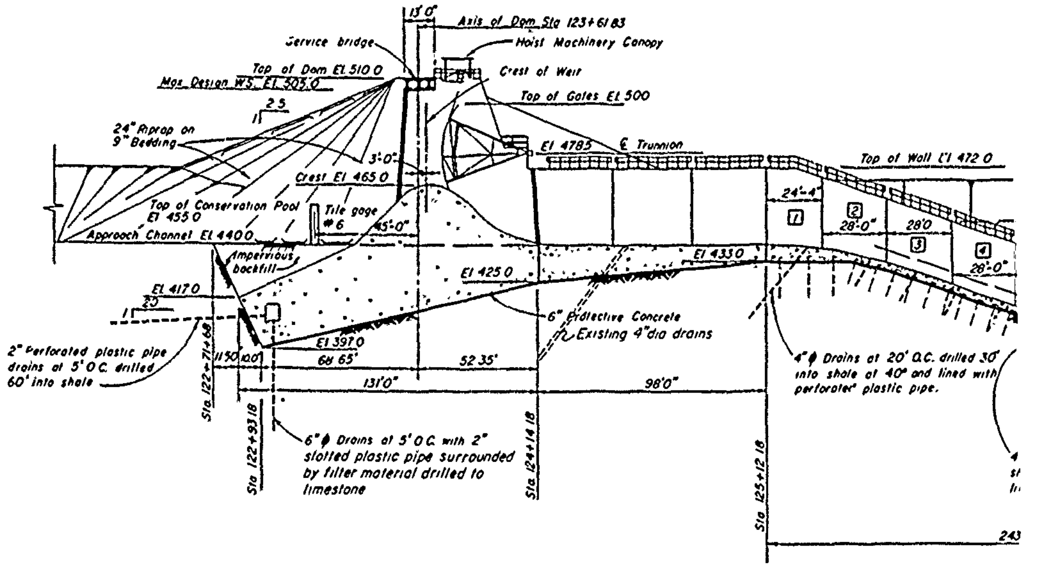


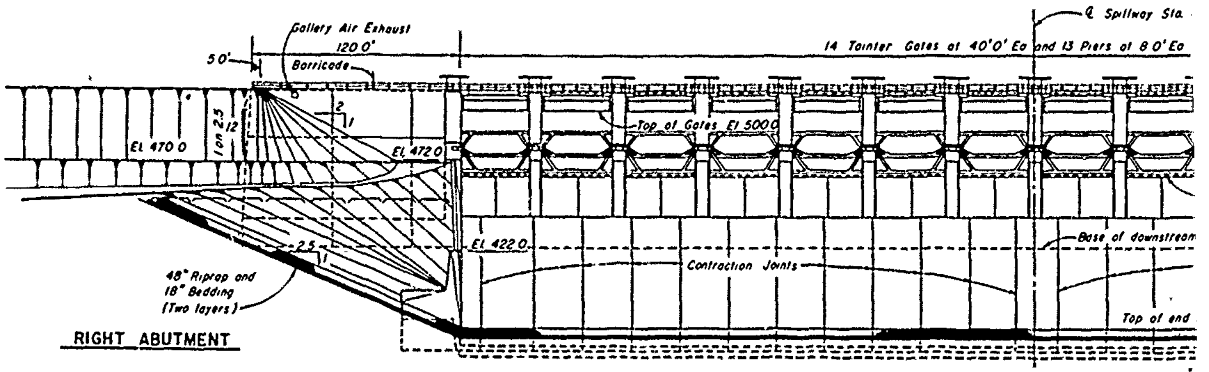
TRANSVERSE SECTION NON-OVERFLOW SECTION
SCALE 1 INCH = 20 FEET



WACO DAM
BOSQUE RIVER, TEXAS

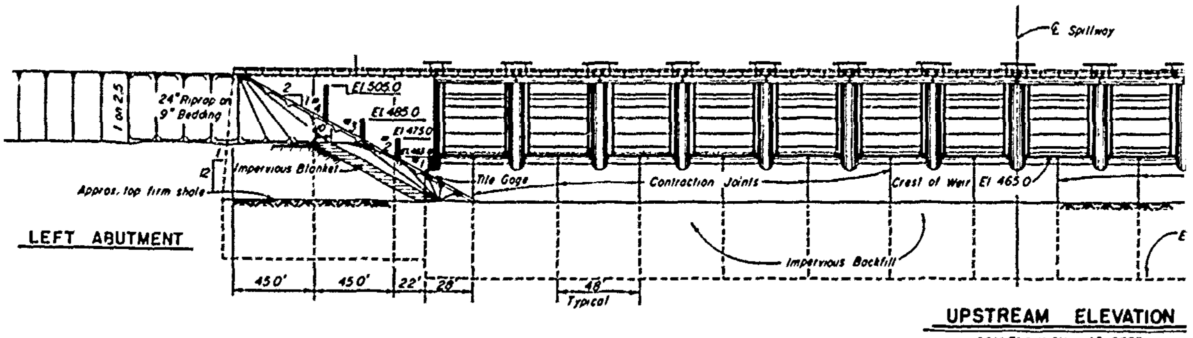
SPILLWAY
PLAN AND SECTIONS





DOWNSTREAM ELEVATI(

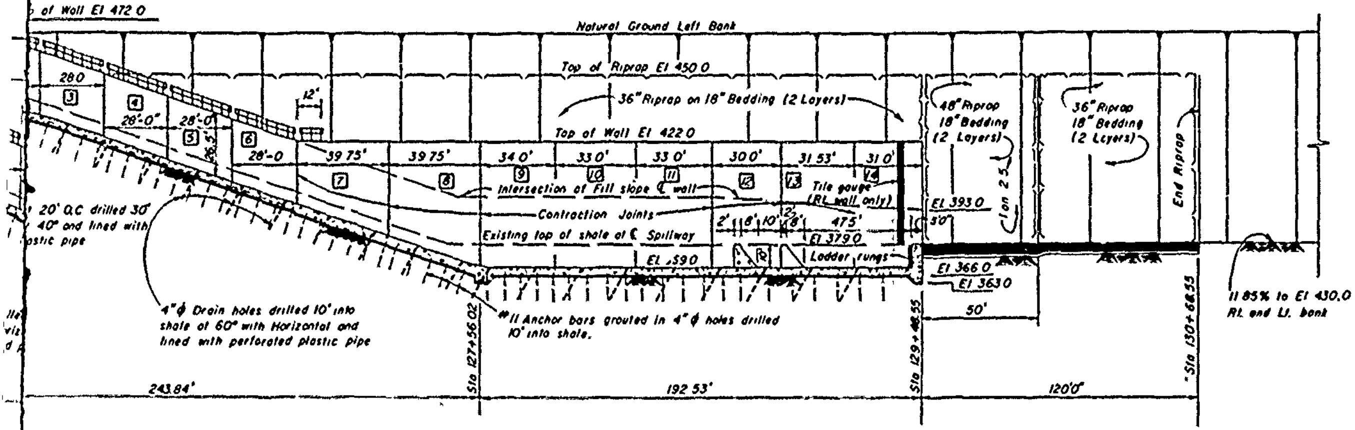
SCALE: 1 INCH = 40 FEET



UPSTREAM ELEVATION

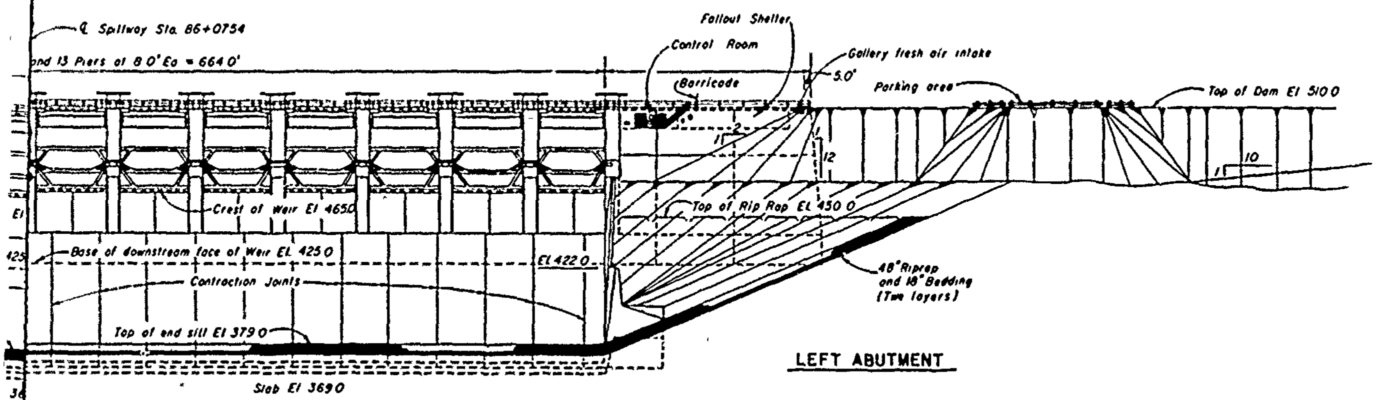
SCALE: 1 INCH = 40 FEET

40 0 40 80



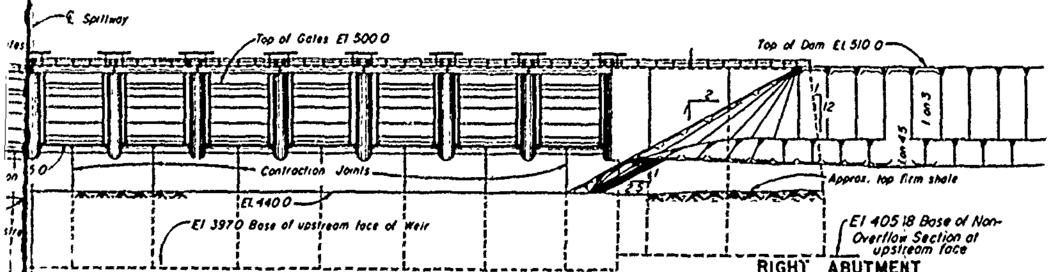
℄ SECTION

SCALE: 1 INCH = 30 FEET



TREAM ELEVATION

SCALE: 1 INCH = 40 FEET



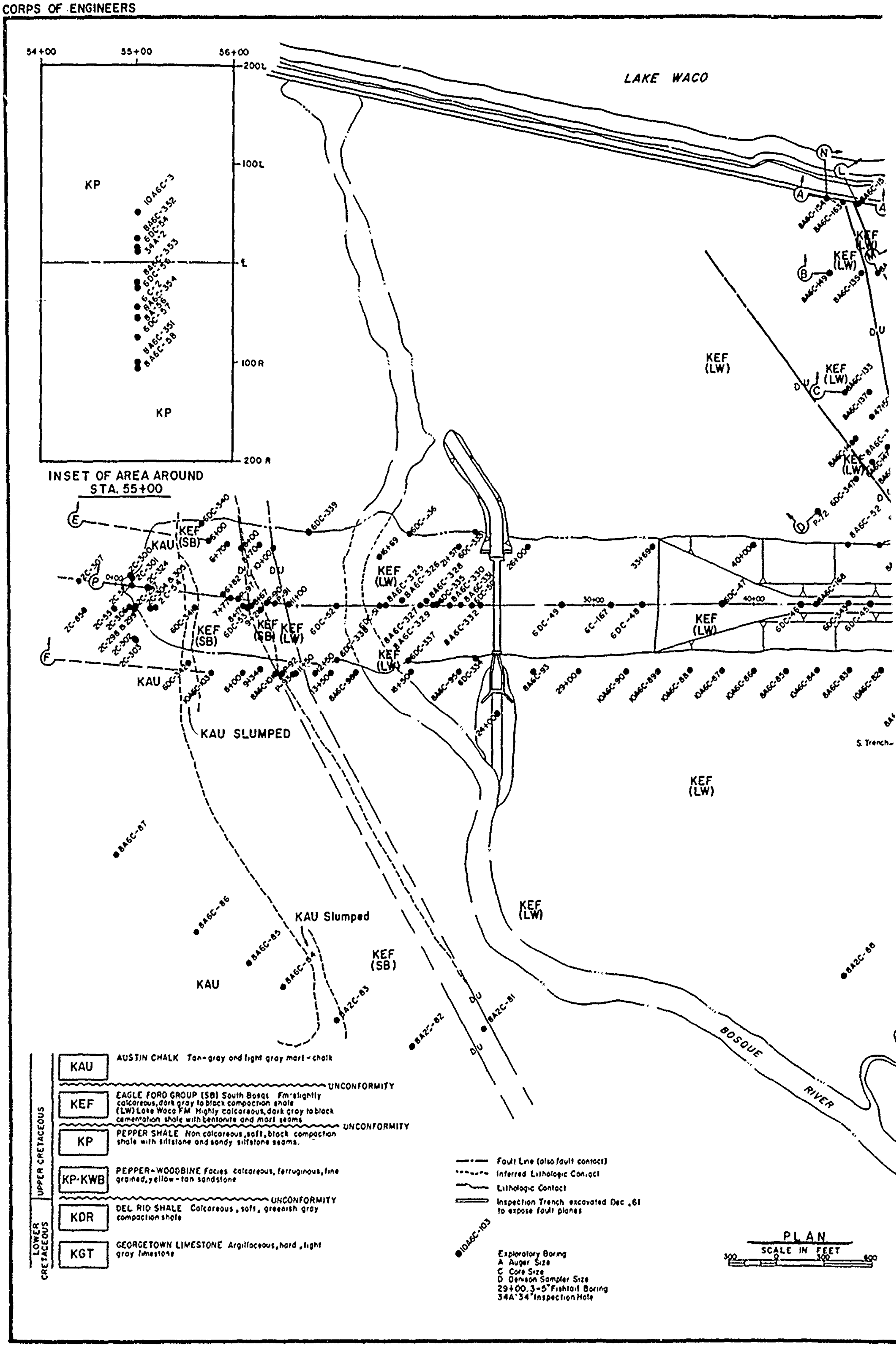
EAM ELEVATION

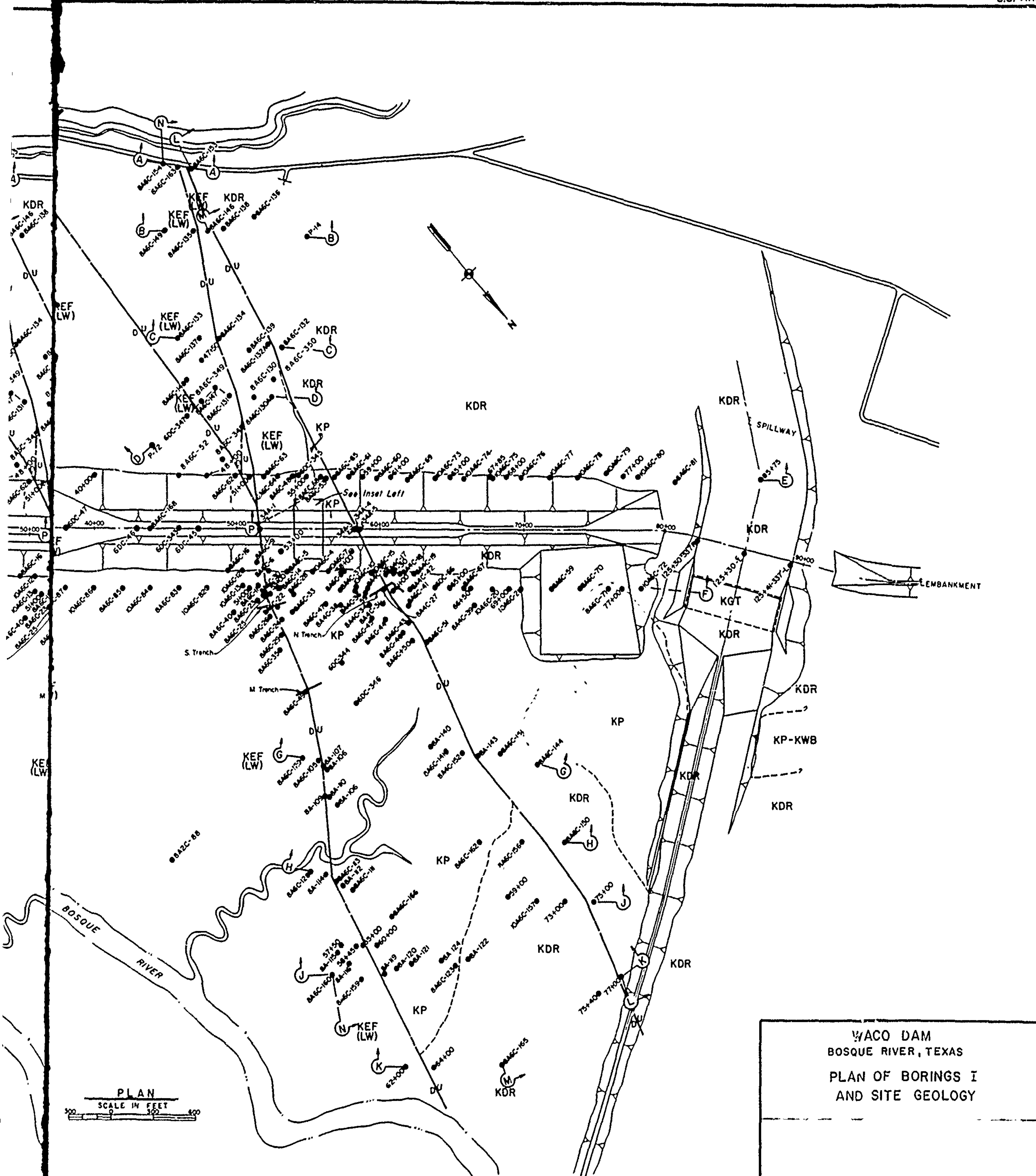
1 INCH = 40 FEET

WACO DAM
BOSQUE RIVER, TEXAS

SPILLWAY
SECTION AND ELEVATIONS

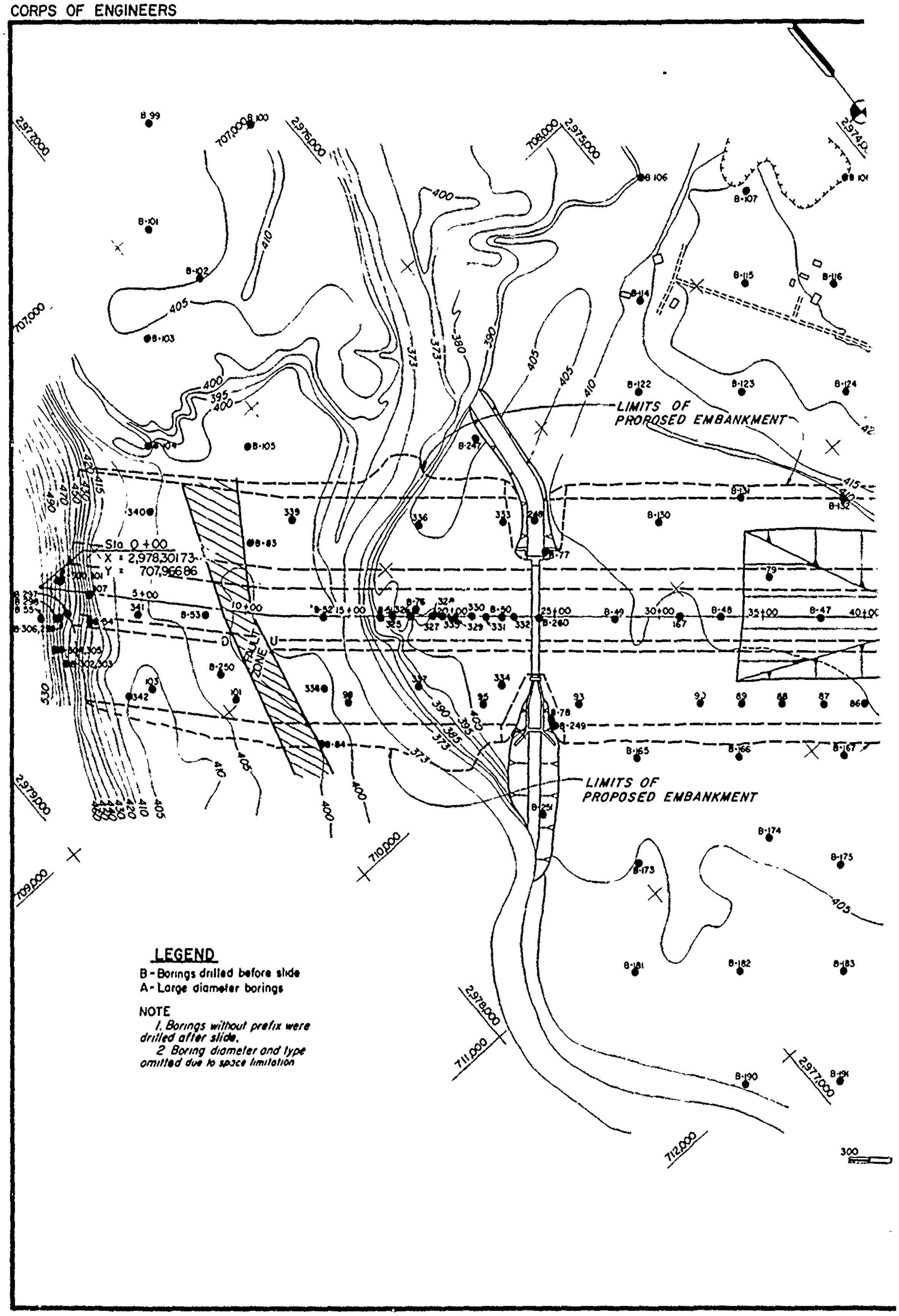
PLATE 10

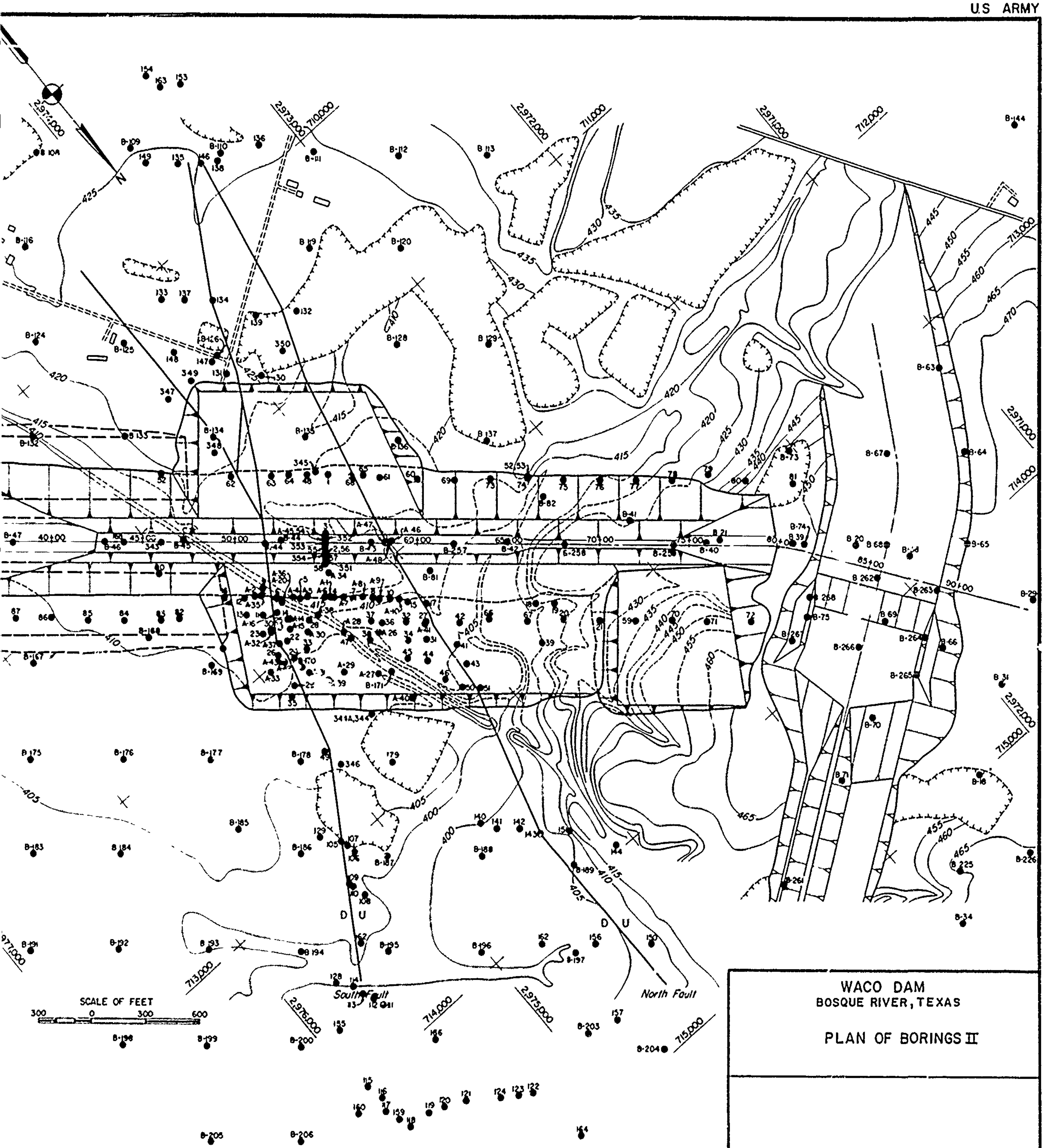

LAKE WACO
INSET OF AREA AROUND STA. 55+00
KP
KEF (LW)
KEF (SB)
KAU
KAU SLUMPED
KAU Slumped
BOSQUE RIVER
UPPER CRETACEOUS
LOWER CRETACEOUS
KAU AUSTIN CHALK Tan-gray and light gray marl-chalk
UNCONFORMITY
KEF EAGLE FORD GROUP (SB) South Bosque Fm slightly calcareous, dark gray to black compaction shale (LW) Lake Waco FM Highly calcareous, dark gray to black cementation shale with bentonite and marl seams
UNCONFORMITY
KP PEPPER SHALE Non calcareous, soft, black compaction shale with siltstone and sandy siltstone seams.
KP-KWB PEPPER-WOODBINE Facies calcareous, ferruginous, fine grained, yellow-tan sandstone
UNCONFORMITY
KDR DEL RIO SHALE Calcareous, soft, greenish gray compaction shale
KGT GEORGETOWN LIMESTONE Argillaceous, hard, light gray limestone
Fault Line (also fault contact)
Inferred Lithologic Contact
Lithologic Contact
Inspection Trench excavated Dec, 61 to expose fault planes
Exploratory Boring
A Auger Size
C Core Size
D Denison Sampler Size
29+00, 3-5" Fishtail Boring
34A 34" Inspection Hole
PLAN
SCALE IN FEET



WACO DAM
BOSQUE RIVER, TEXAS

PLAN OF BORINGS I
AND SITE GEOLOGY

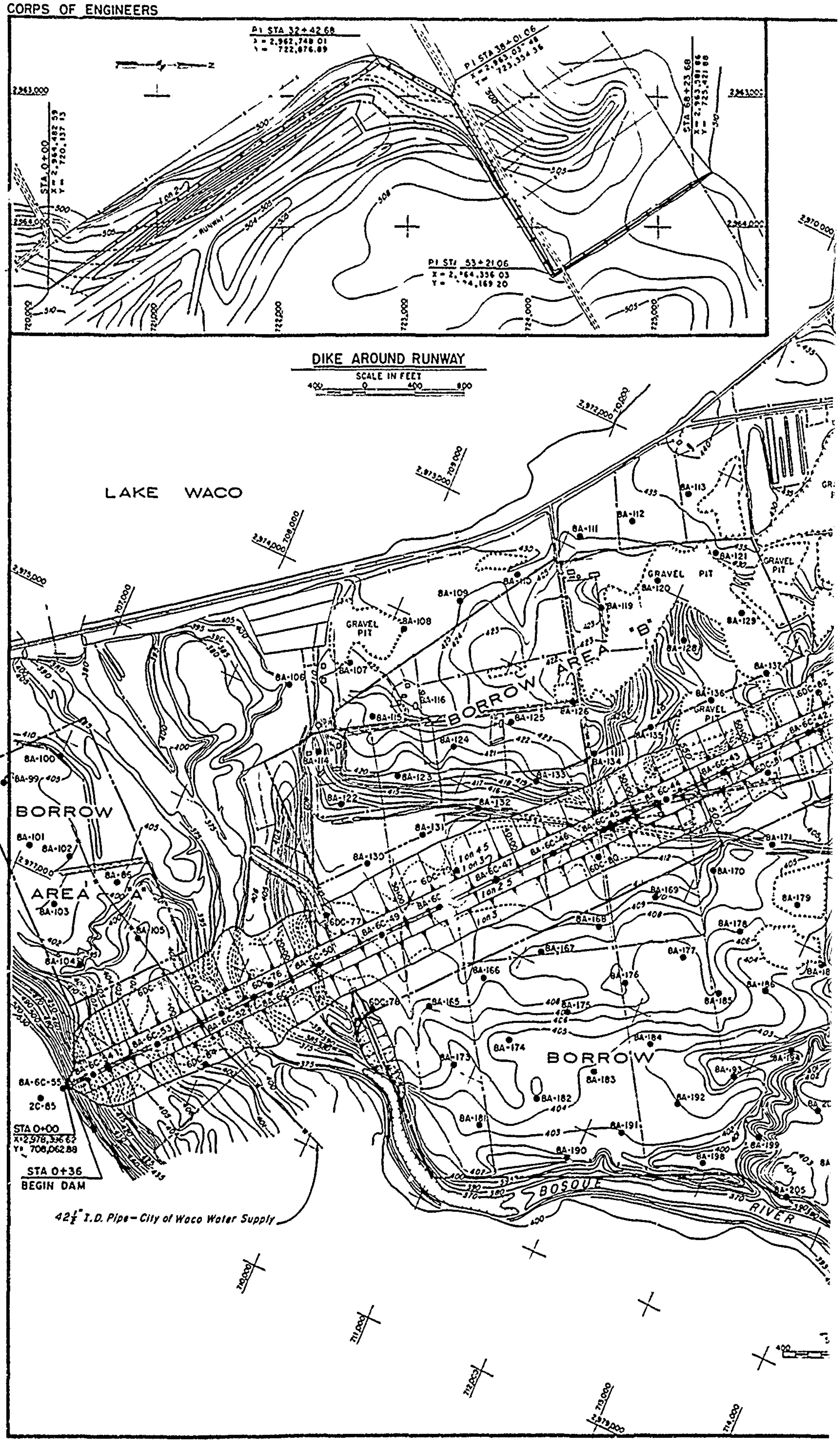

LIMITS OF PROPOSED EMBANKMENT
LIMITS OF PROPOSED EMBANKMENT
FAULT ZONE
Sta 0+00
X = 2,978,301.73
Y = 707,966.86
LEGEND
B - Borings drilled before slide
A - Large diameter borings
NOTE
1. Borings without prefix were drilled after slide.
2. Boring diameter and type omitted due to space limitation



WACO DAM
BOSQUE RIVER, TEXAS

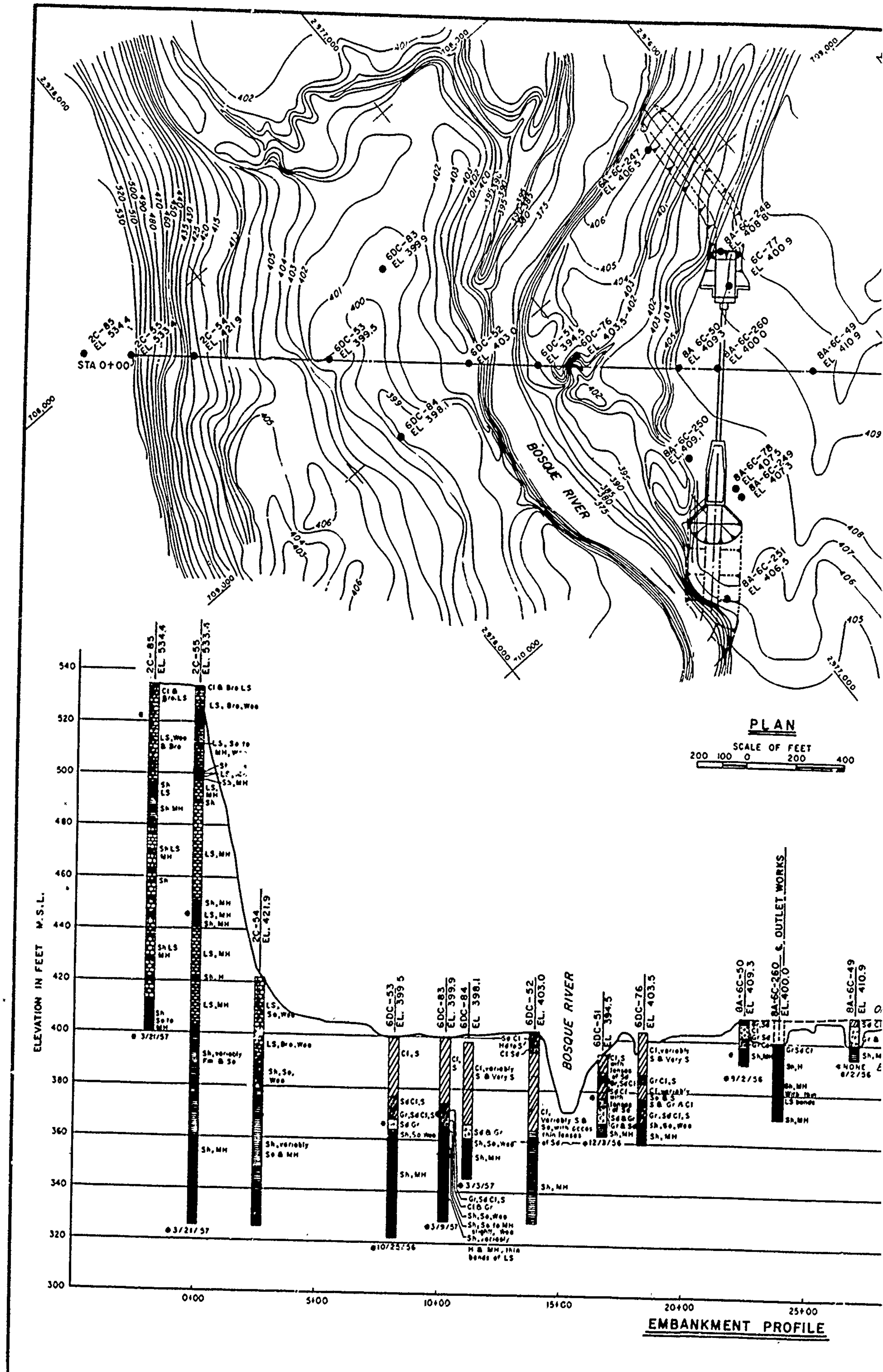
PLAN OF BORINGS II

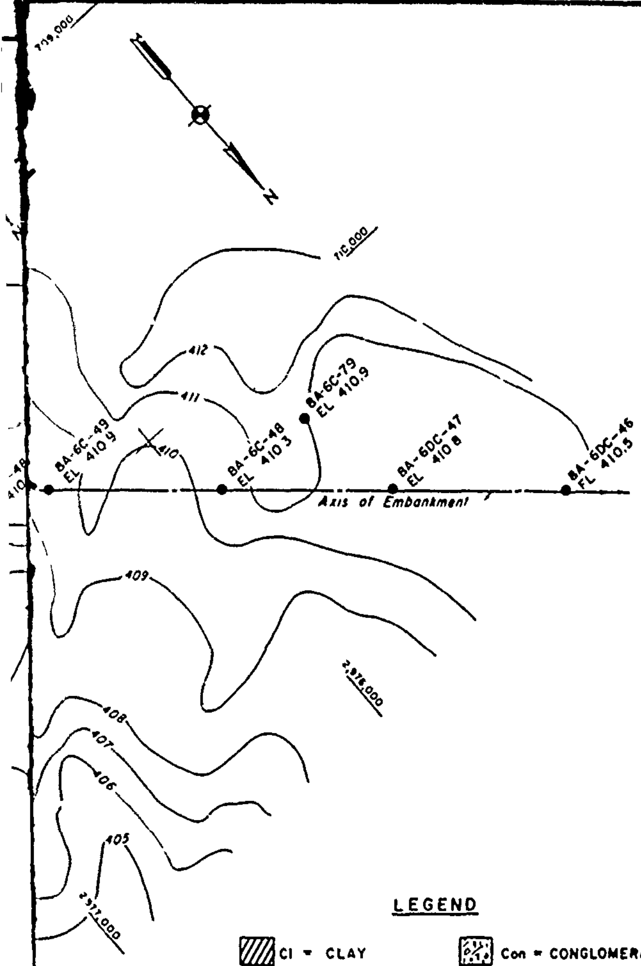
CORPS OF ENGINEERS

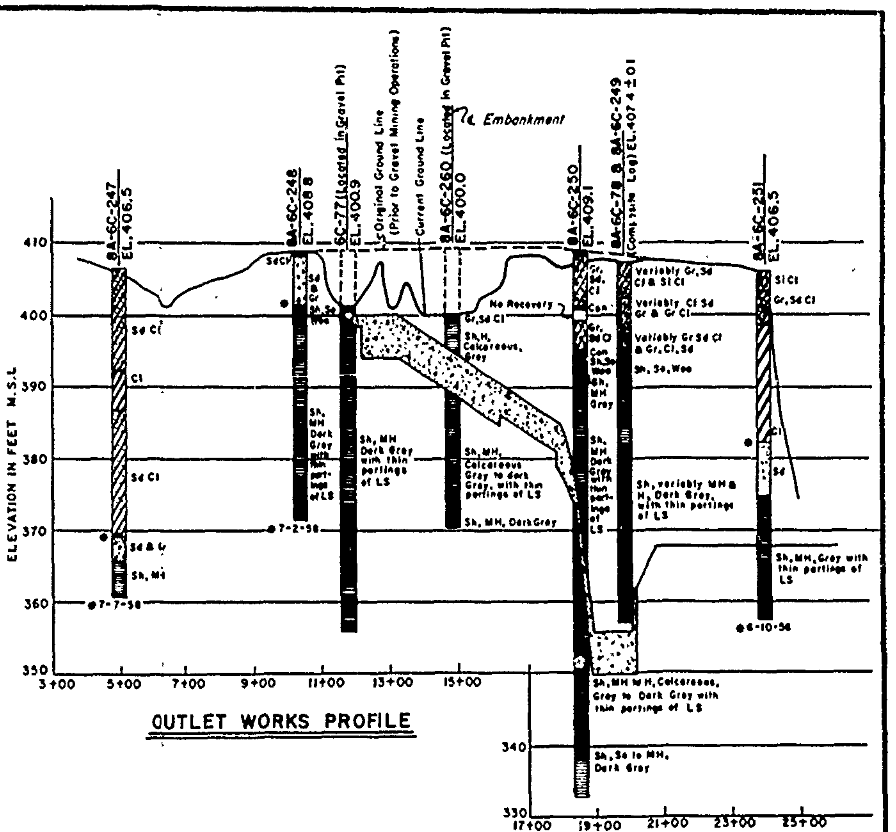



U. S. ARMY
BORROW AREA "C"
BORROW AREA "F"
BORROW AREA "E"
AREA "D"
RIVER
RUNWAY
APRON
AIRPORT ROAD
GRAVEL PIT
LEGEND
2C - 2" CORE BORING
6C - 6" CORE BORING
6D - 6" DENISON BARREL BORING
8A - 8" AUGER BORING
PLAN
SCALE IN FEET
BRAZOS RIVER BASIN, TEXAS
WACO RESERVOIR
BOSQUE RIVER, TEXAS
PLAN OF BORINGS III
SCALE AS SHOWN
FORT WORTH DISTRICT FORT WORTH, TEXAS JUNE 1958

PLATE 13


PLAN
SCALE OF FEET
200 100 0 200 400
BOSQUE RIVER
STA 0+00
2C-85 EL 534.4
2C-55 EL 533.5
2C-54 EL 421.9
6DC-53 EL 399.5
6DC-83 EL 399.9
6DC-84 EL 398.1
6DC-52 EL 403.0
6DC-51 EL 394.5
6DC-76 EL 403.5
8A-6C-50 EL 409.3
8A-6C-260 EL 400.0
8A-6C-49 EL 410.9
8A-6C-247 EL 406.5
8A-6C-248 EL 408.8
6C-77 EL 400.9
8A-6C-250 EL 409.1
8A-6C-78 EL 407.5
8A-6C-249 EL 407.3
8A-6C-251 EL 406.3
℄ OUTLET WORKS
ELEVATION IN FEET M.S.L.
540
520
500
480
460
440
420
400
380
360
340
320
300
0+00
5+00
10+00
15+00
20+00
25+00
EMBANKMENT PROFILE





OUTLET WORKS PROFILE

LEGEND

- Cl = CLAY
- Si = SILT
- Sd = SAND
- Gr = GRAVEL
- Con = CONGLOMERATE
- LS = LIMESTONE
- LS = WEATHERED LIMESTONE
- Sh = SHALE
- Sh = WEATHERED SHALE

VISUAL CLASSIFICATION ABBREVIATIONS

Gr	= Gravel or Gravelly	Fm	= Firm
Sd	= Sand or Sandy	H	= Hard
Si	= Silt or Silty	MH	= Medium Hard
Cl	= Clay or Clayey	S	= Stiff
LS	= Limestone	So	= Soft
Sh	= Shale or Shaly	Bro	= Broken
Con	= Conglomerate	Wea	= Weathered

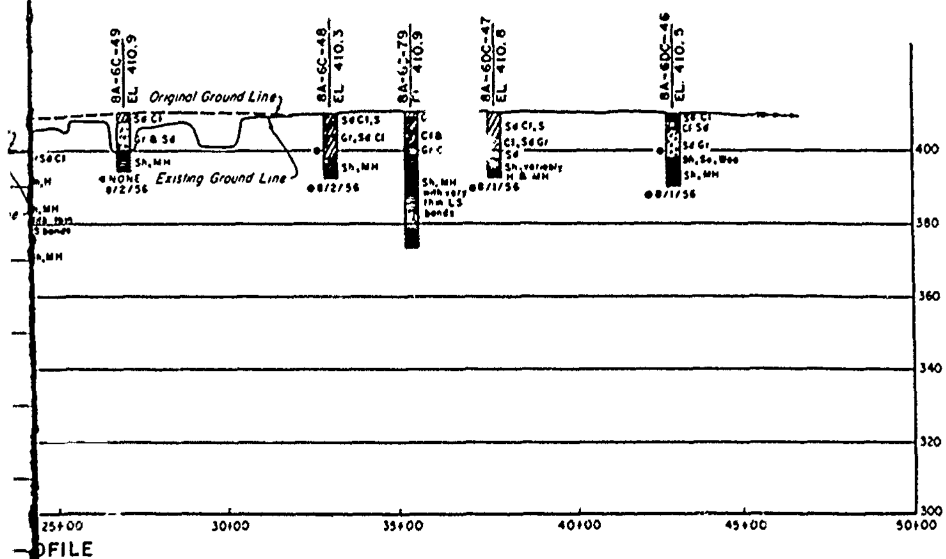
Example: Gr, Sd Cl, So = Gravelly, Sandy Clay, Soft.

● = Denotes Ground Water Level

● 8/2/56 = Denotes date ground water level (If any) was measured.

8A = 8" Dia. Machine Auger Boring.
6C = 6" Dia. Core Boring.
2C = 2" Dia. Core Boring
6D = 6" Denison Barrel Boring.

EXAMPLE.
8A-6C-248 = Machine Augered thru overburden or unconsolidated material; 6" Core into Primary Strata or consolidated material; Boring No 248

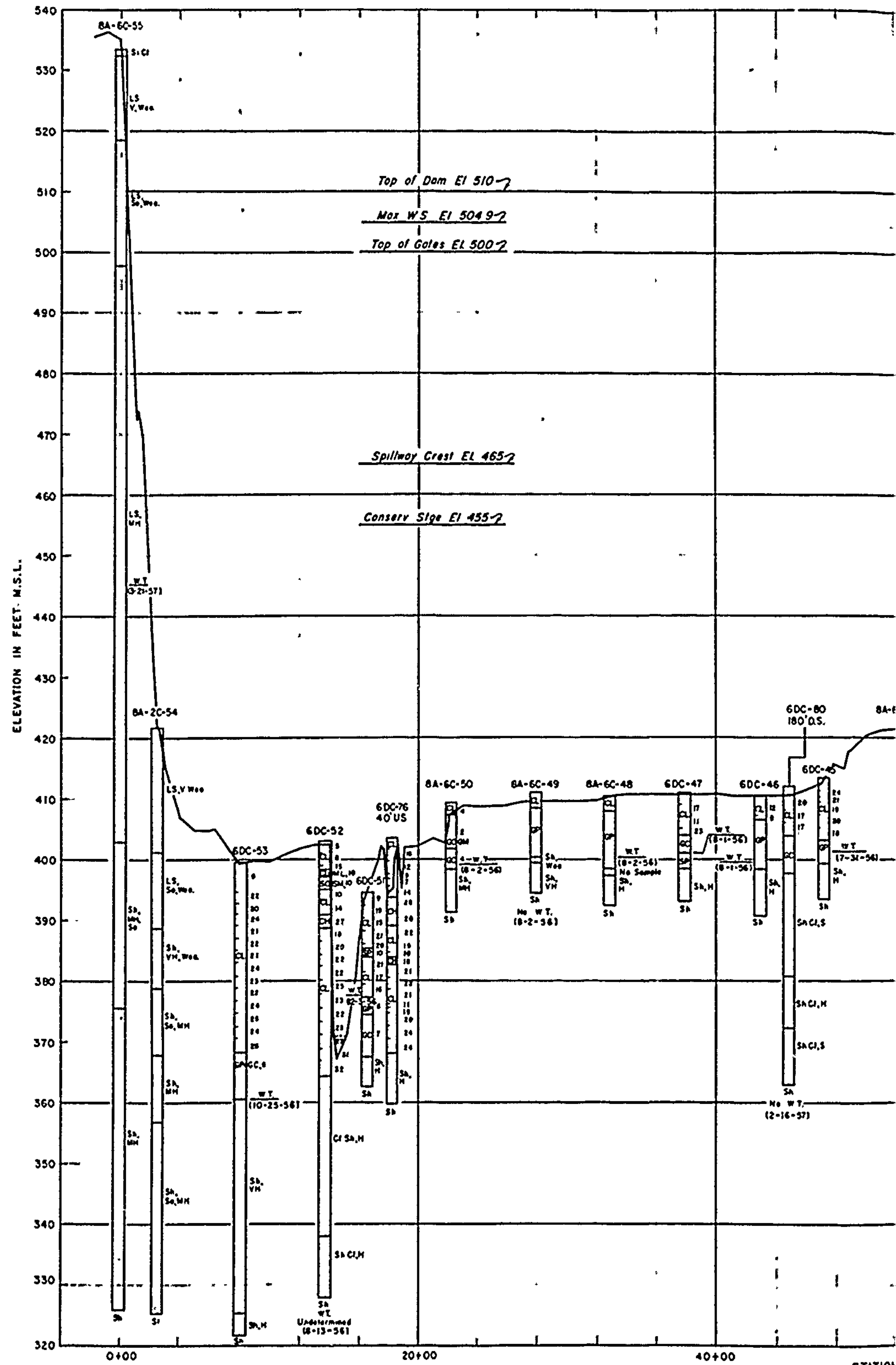


PROFILE

NOTE
The absence of ground water level determinations on the graphic boring logs does not mean necessarily that ground water will not be encountered at the locations or within the vertical reaches of the borings.

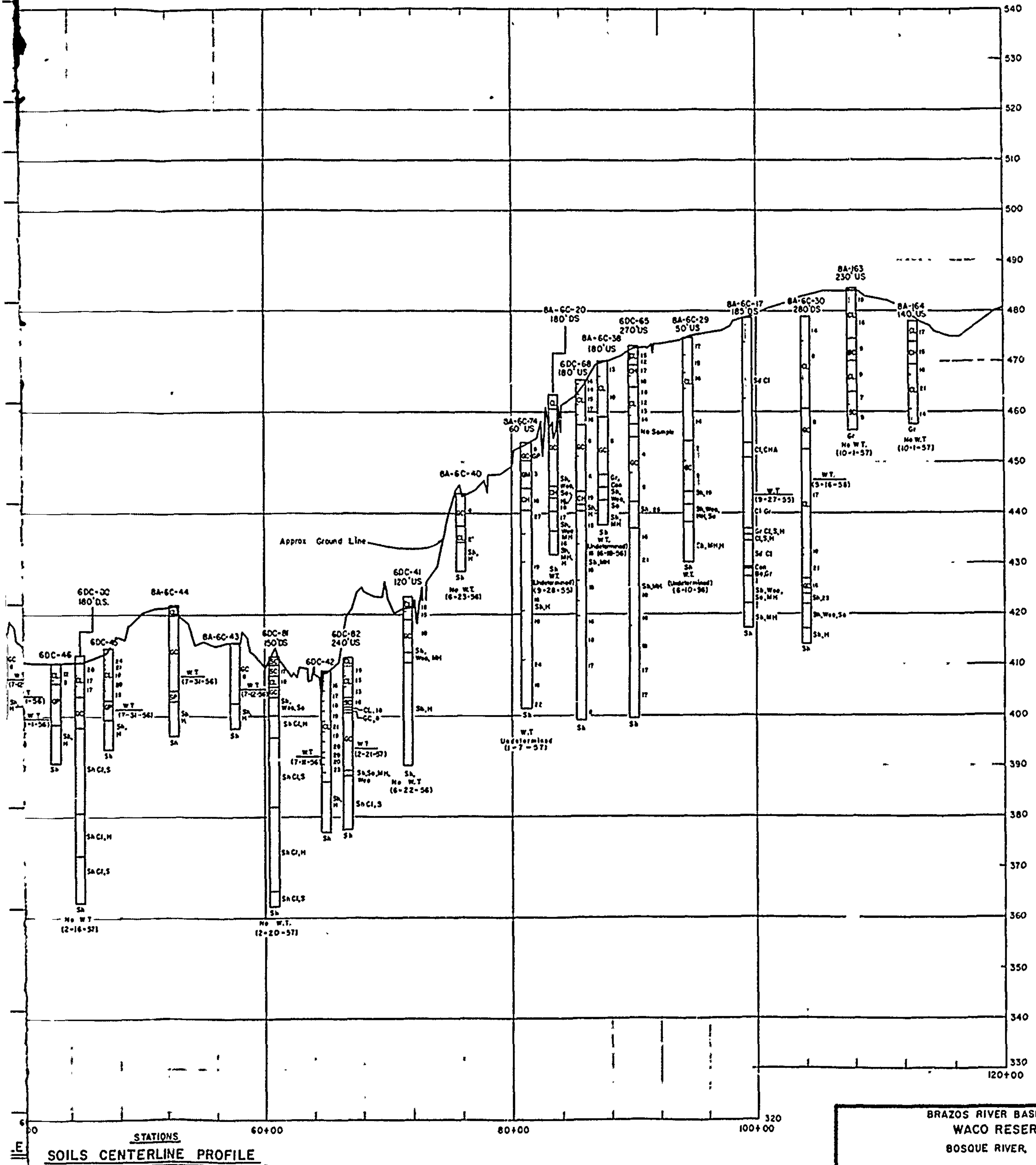
WACO DAM
BOSQUE RIVER, TEXAS

OUTLET WORKS
BORING PLAN AND PROFILE



SOILS CENTERLI

U S ARMY



BRAZOS RIVER BASIN, TEXAS
WACO RESERVOIR
BOSQUE RIVER, TEXAS

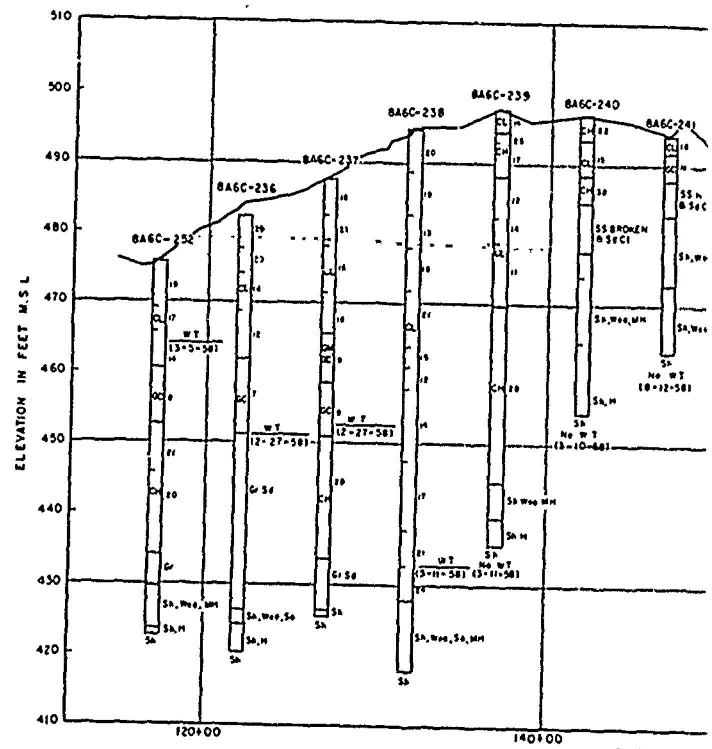
FOUNDATION ℄ PROFILE I
BEFORE SLIDE

SCALES AS SHOWN

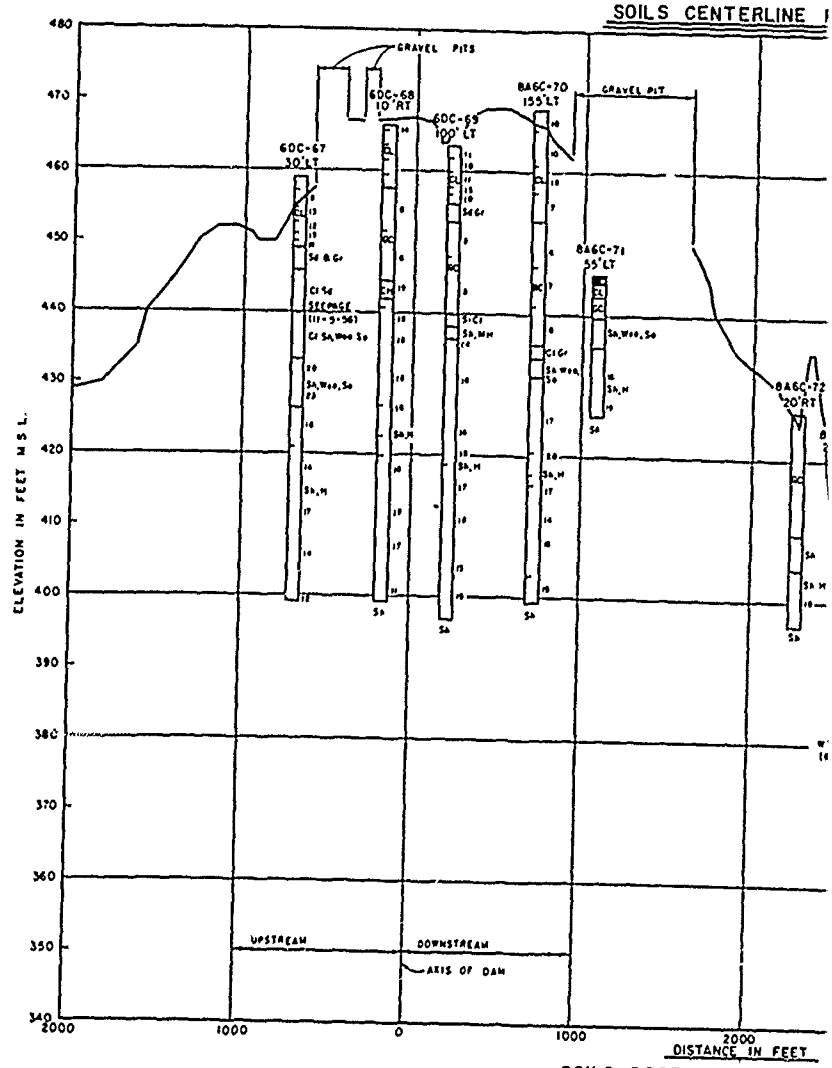
FORT WORTH DISTRICT FORT WORTH, TEXAS JUNE 1958

PLATE 15

CORPS OF ENGINEERS

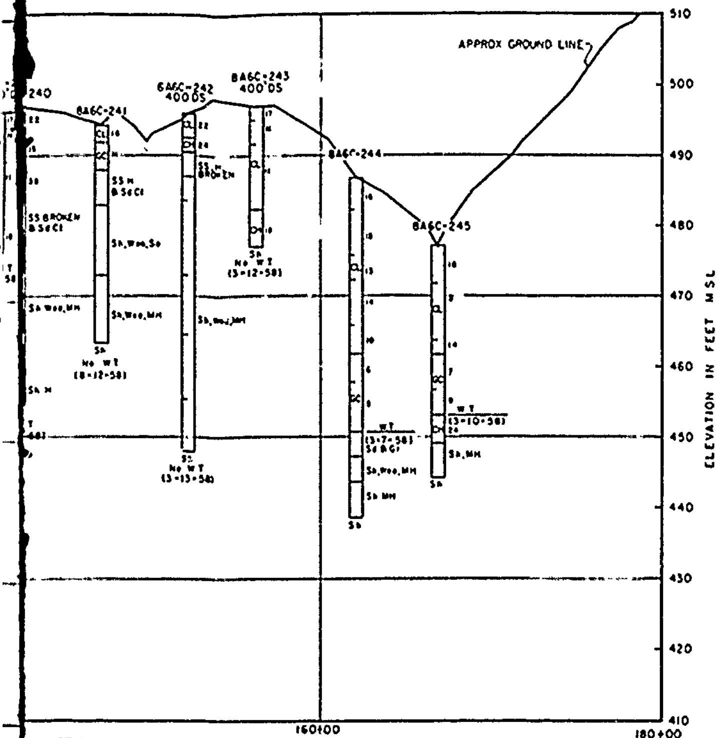


SOILS CENTERLINE



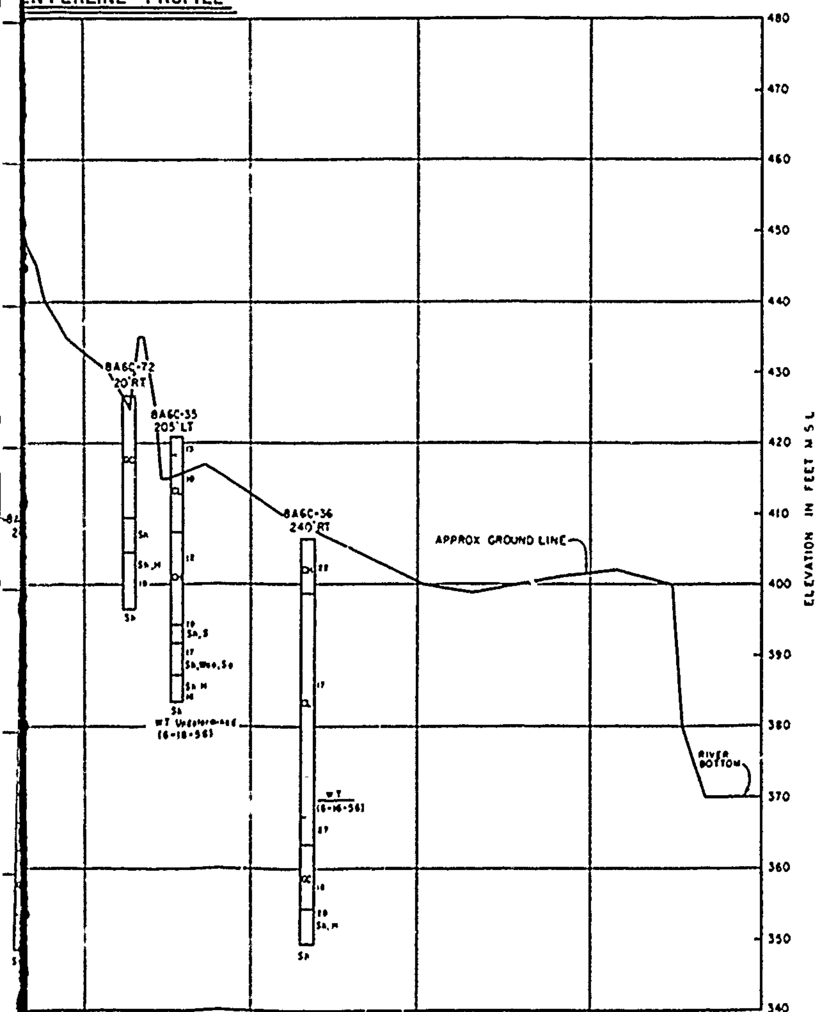
SOILS PROFILE - OUTLET

U.S. ARMY



STATIONS

ENTERLINE PROFILE



STANCE IN FEET

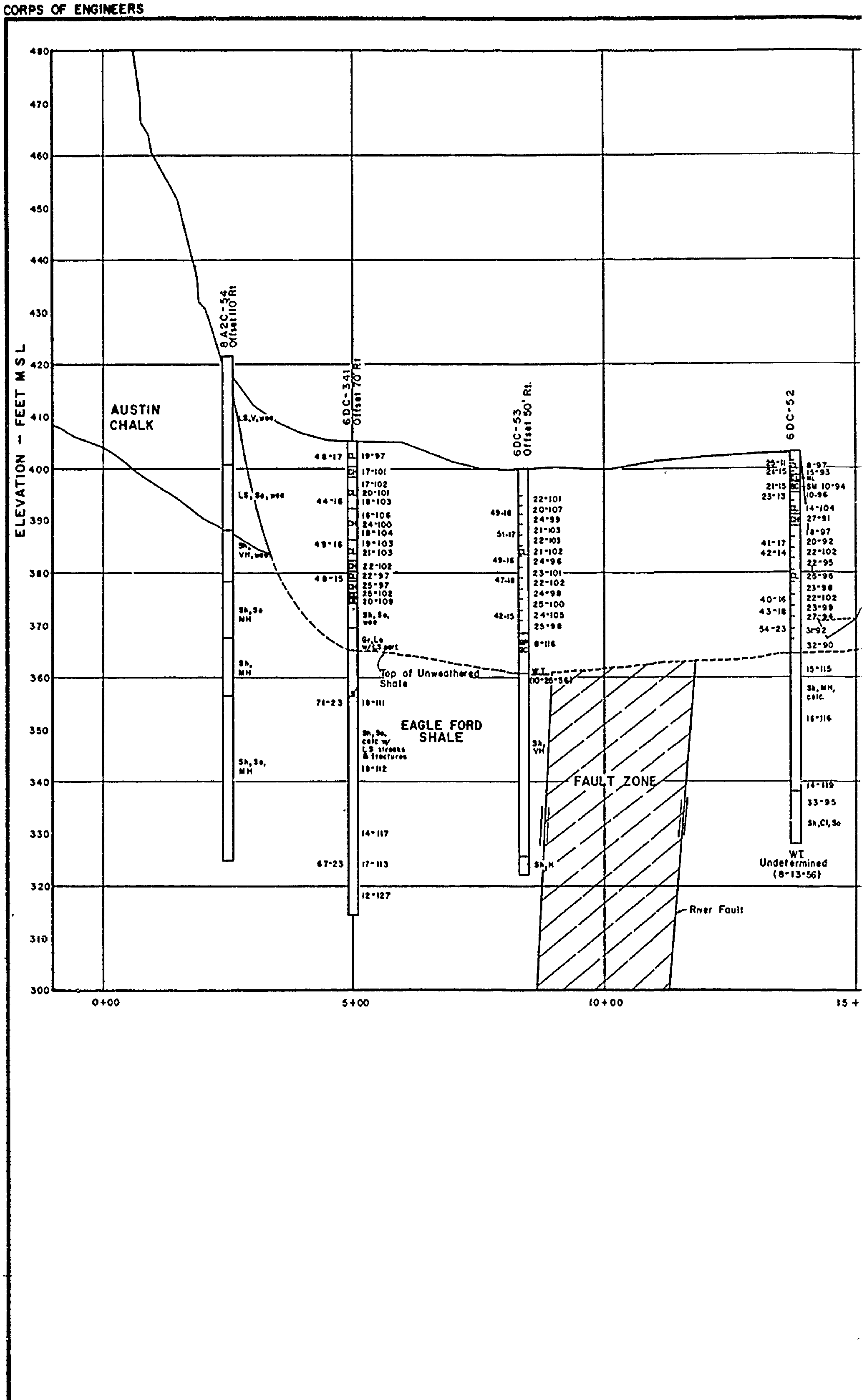
LE-OUTLET CHANNEL

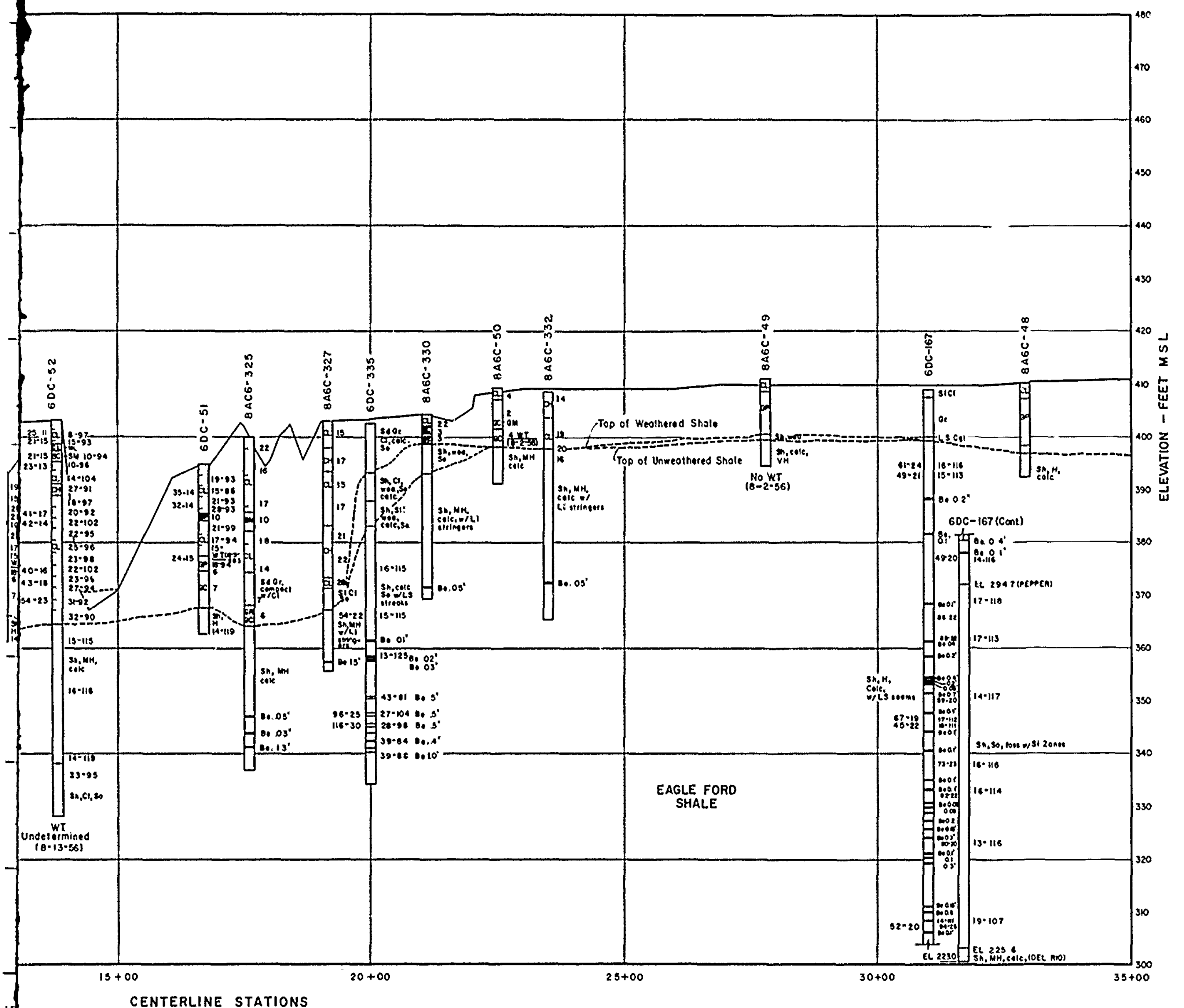
BRAZOS RIVER BASIN, TEXAS
WACO RESERVOIR
BOSQUE RIVER, TEXAS

FOUNDATION ℄ PROFILE II
AND OUTLET WORKS ℄ PROFILE
BEFORE SLIDE

FORT WORTH DISTRICT FORT WORTH, TEXAS JUNE 1958

PLATE 16

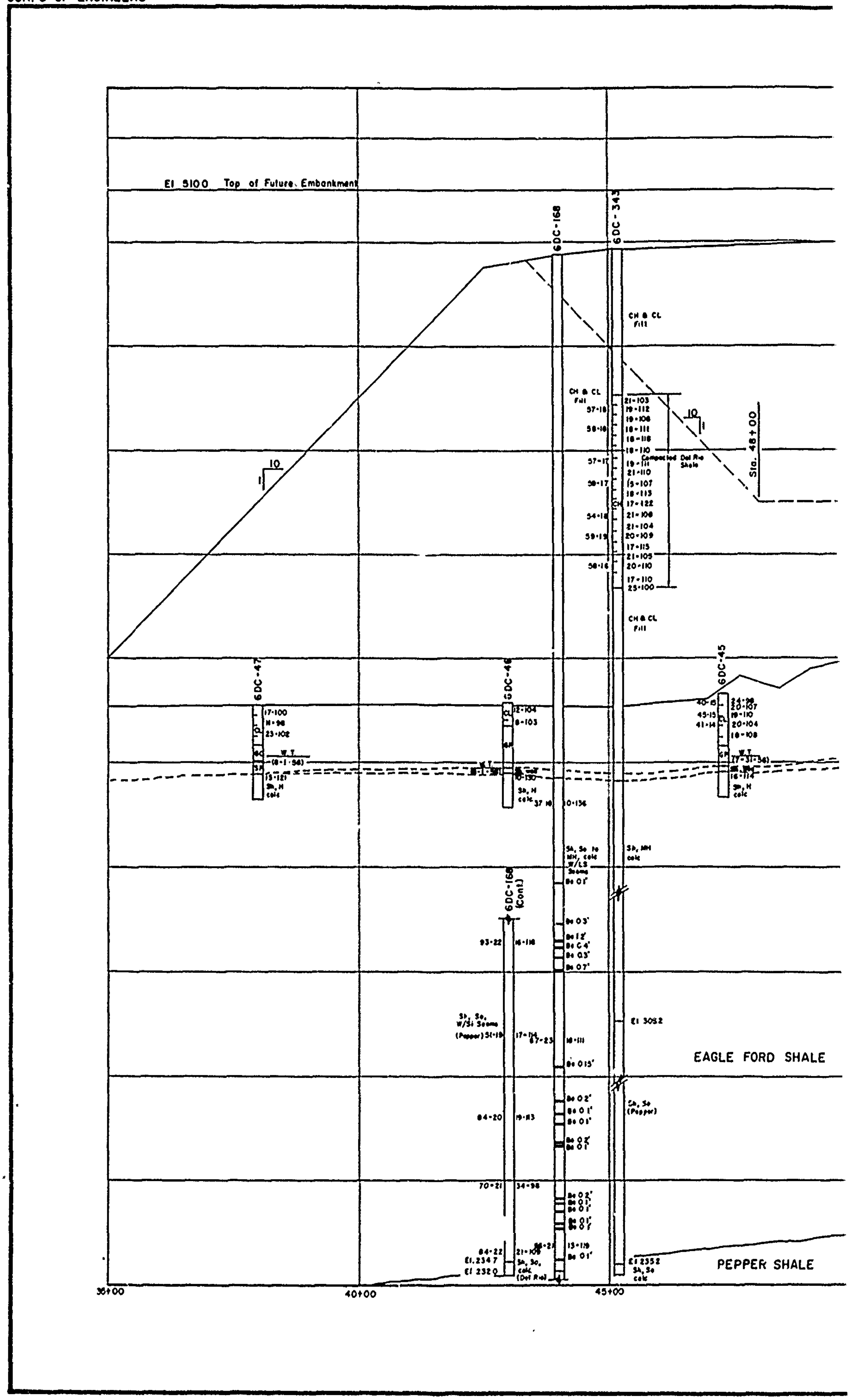

ELEVATION - FEET MSL
AUSTIN CHALK
EAGLE FORD SHALE
FAULT ZONE
Top of Unweathered Shale
River Fault
8A2C-54 Offset 110' Rt.
6DC-341 Offset 70' Rt.
6DC-53 Offset 50' Rt.
6DC-52
W.T. (10-25-56)
WT Undetermined (8-13-56)
0+00
5+00
10+00
15+

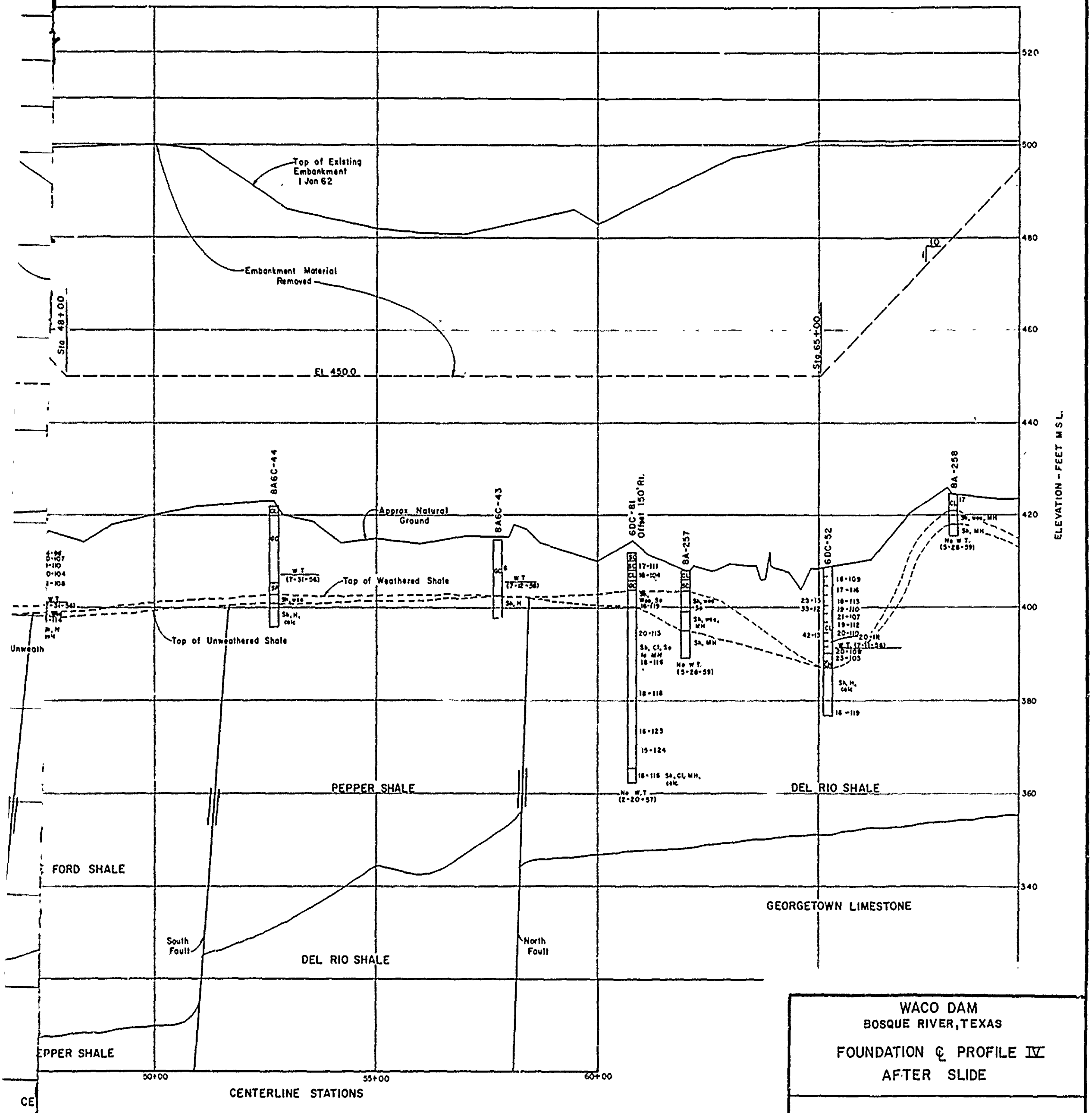


WACO DAM
BOSQUE RIVER, TEXAS

FOUNDATION ℄ PROFILE III
AFTER SLIDE

CORPS OF ENGINEERS

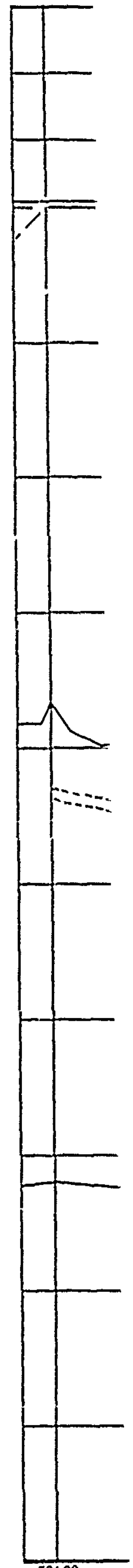




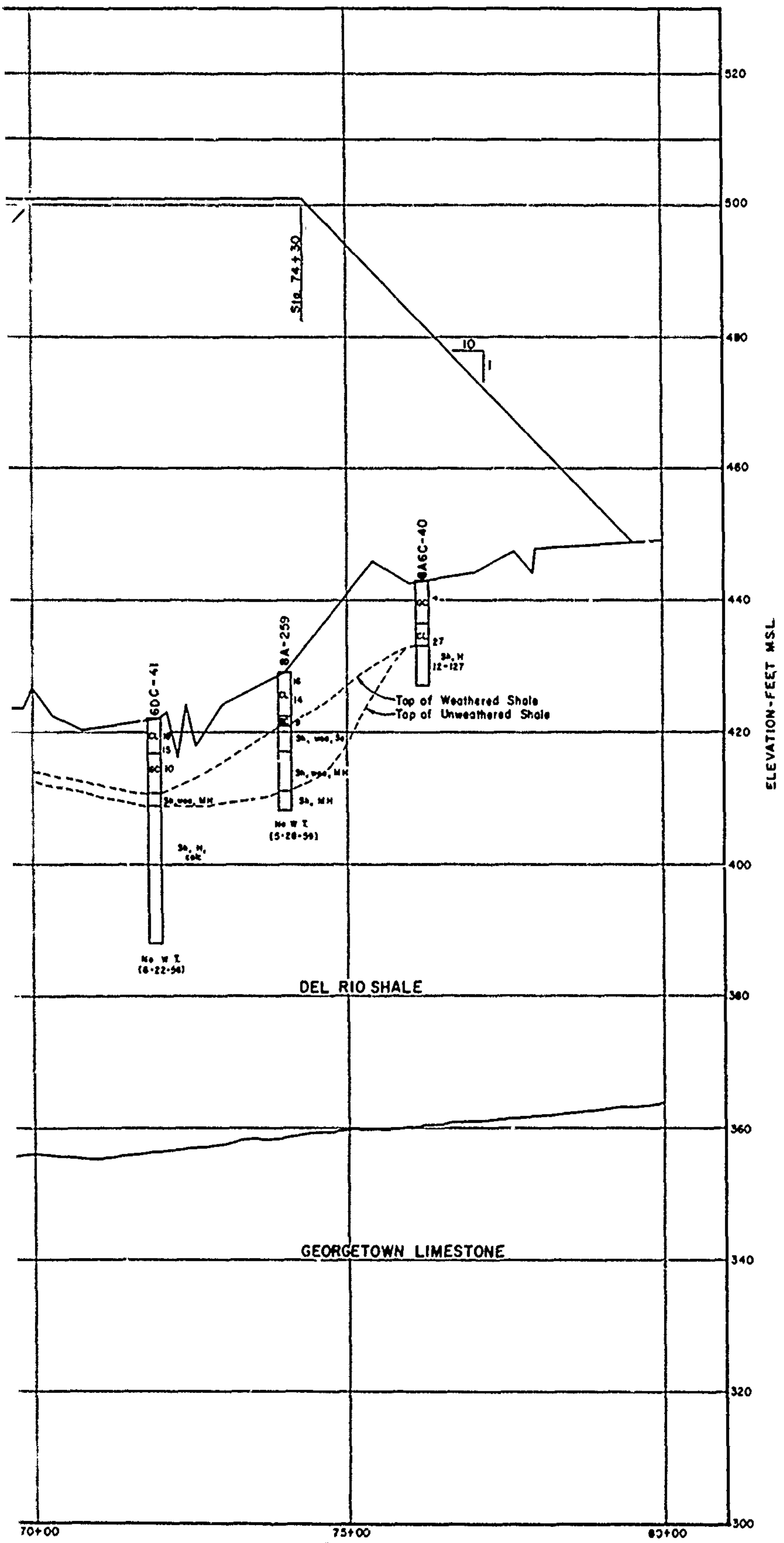
WACO DAM
BOSQUE RIVER, TEXAS

FOUNDATION ℄ PROFILE IV
AFTER SLIDE

CORPS OF ENGINEERS



70+00



LEGEND

LABORATORY CLASSIFICATION

GP Poorly-graded gravels, gravel-sand mixtures, little or no fines.
GM Silty gravels, gravel-sand-silt mixtures.
GC Clayey gravels, gravel-sand-clay mixtures.
SP Poorly-graded sands, gravelly sands, little or no fines
SM Silty sands, sand-silt mixtures.
SC Clayey sands, sand-clay mixtures.
ML Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
CL Inorganic clays of low plasticity, gravelly clays, sandy clays, silty clays, lean clays
CH Inorganic clays of high plasticity, fat clays.

VISUAL CLASSIFICATION

Sd Sand or Sandy
Cl Clay or Clayey
Si Silt or Silty
Sh Shale or Shaly
Gr Gravel or Gravelly
Li Lime or Limy
Cgl Conglomerate
Co Cobbles
LS Limestone
SS Sandstone
Sms Seams
Be Bentonite
Frag Fragments
S Slickensided
Calc Calcareous
Foss Fossiliferous
Sl Slightly
Lo Loose
D Dense
So Soft
V Very Soft
MH Medium Hard
H Hard
VH Very Hard
Wea Weathered
Unwea Unweathered
W.T. Water Table
No W.T. No Water Table
W. T. Undet Water Table Undetermined

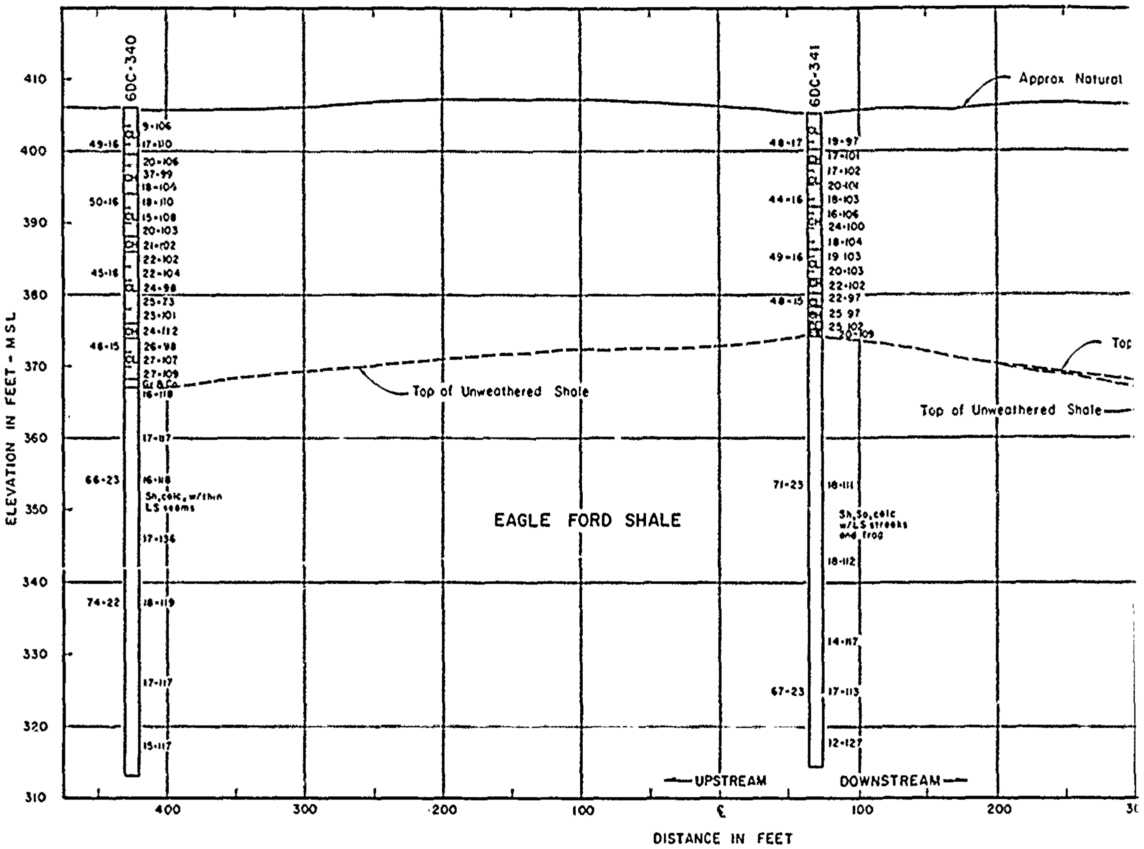
NOTES:

a. Absence of ground water levels opposite boring logs does not necessarily mean that ground water will not be encountered at the locations or within the vertical reaches of the borings.
b. Figures to the left of boring logs are the Atterberg limits. The first figure is the liquid limit and the second figure is the plastic limit.
c. The first figure to the right of boring logs is the water content in percent of the dry weight, and the second figure is the natural dry density in pounds per cubic foot.
d. The number to the right of Bentonite is the the thickness of the Bentonite seam.

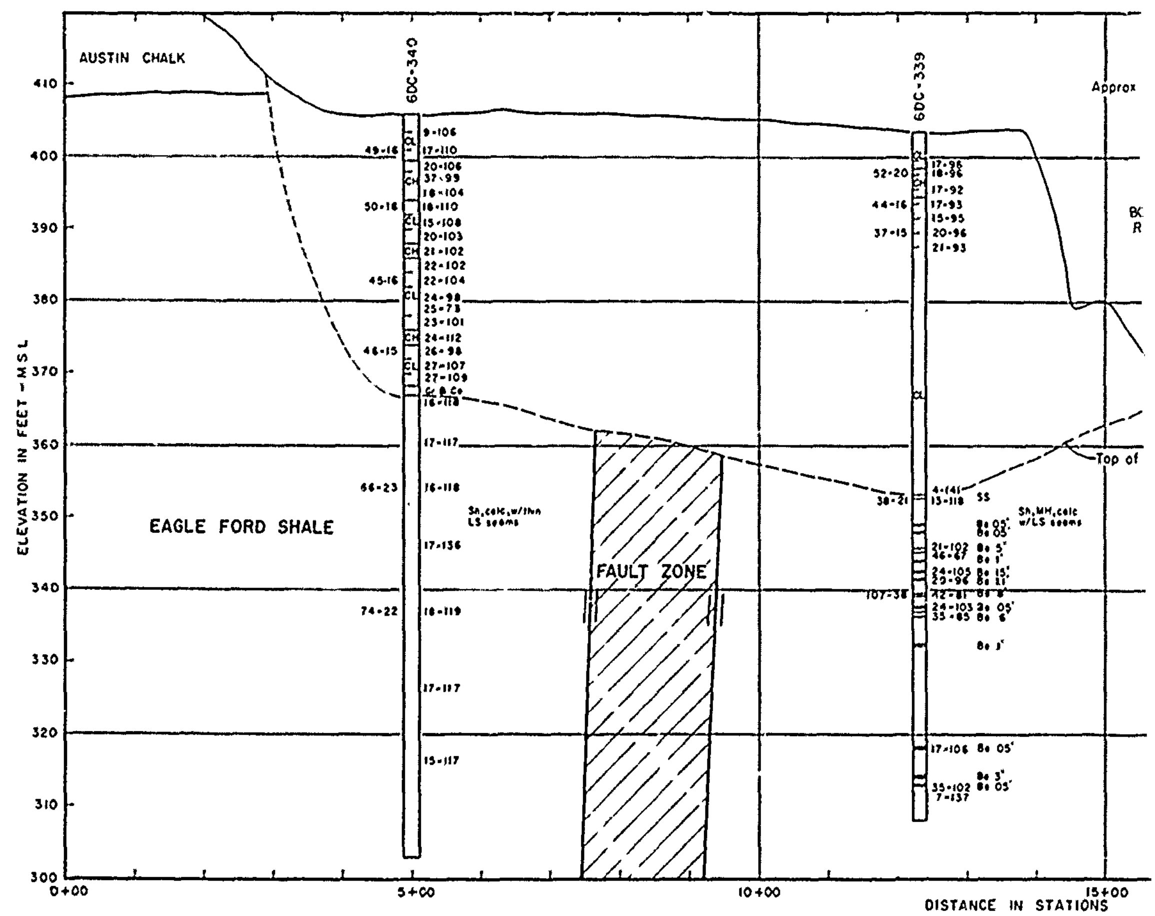
WACO DAM
BOSQUE RIVER, TEXAS

FOUNDATION ℄ PROFILE V
AFTER SLIDE

CORPS OF ENGINEERS

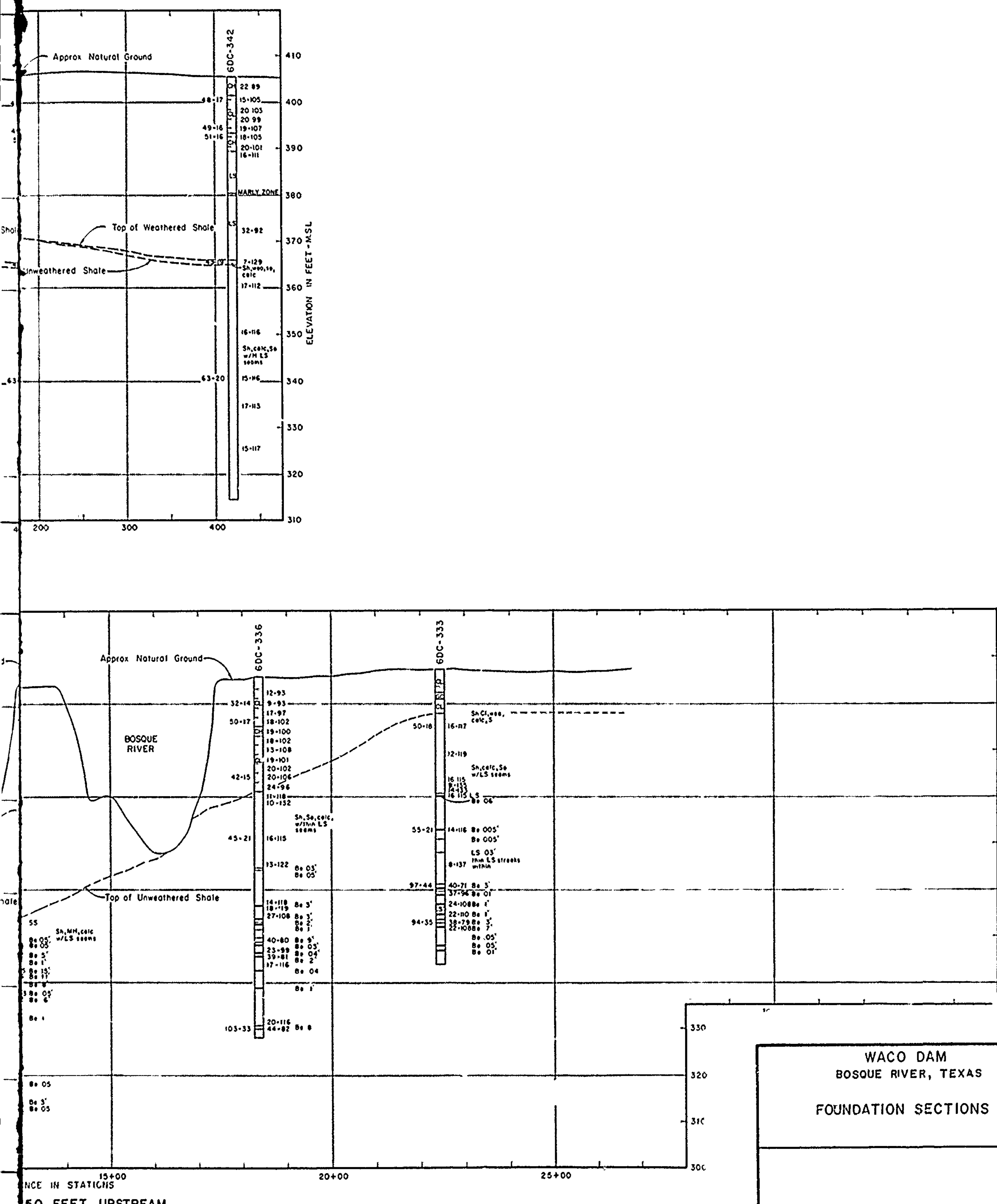


SECTION AT STATION 5+00



SECTION 450 FEET UPSTR

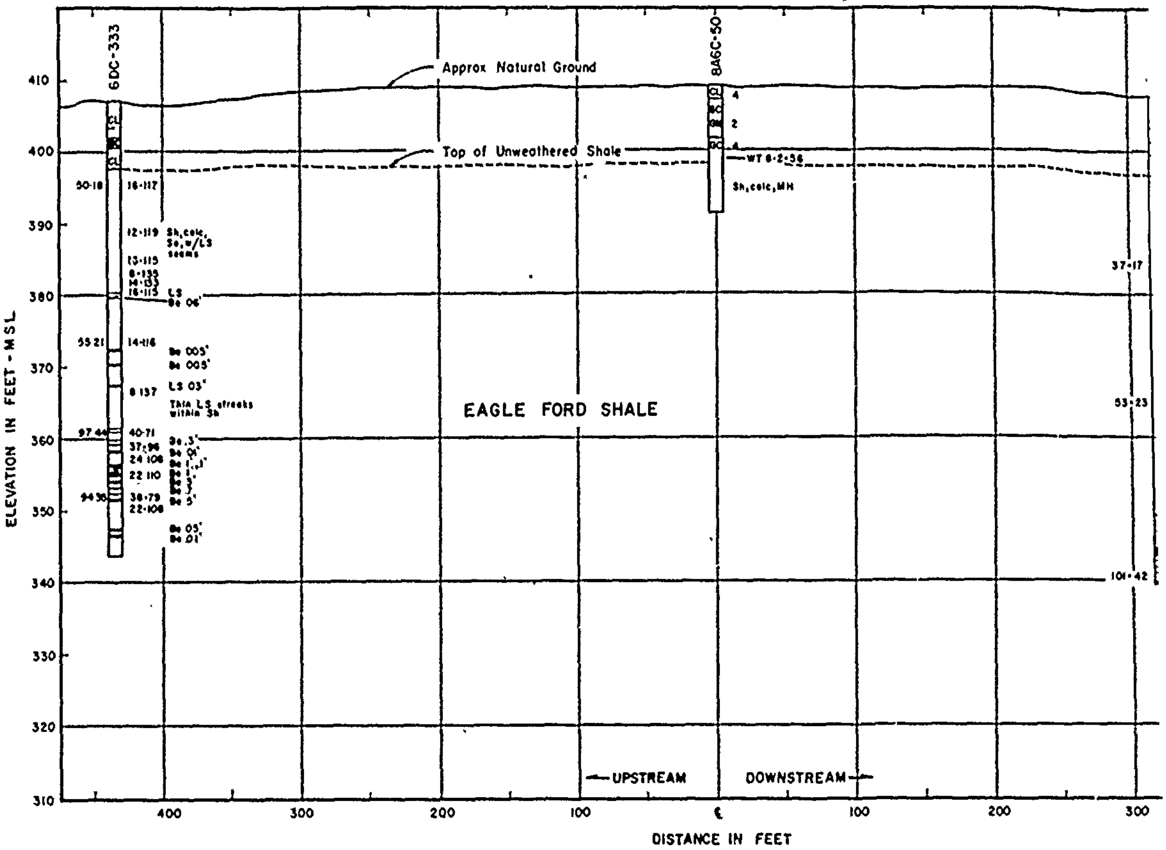
US ARMY



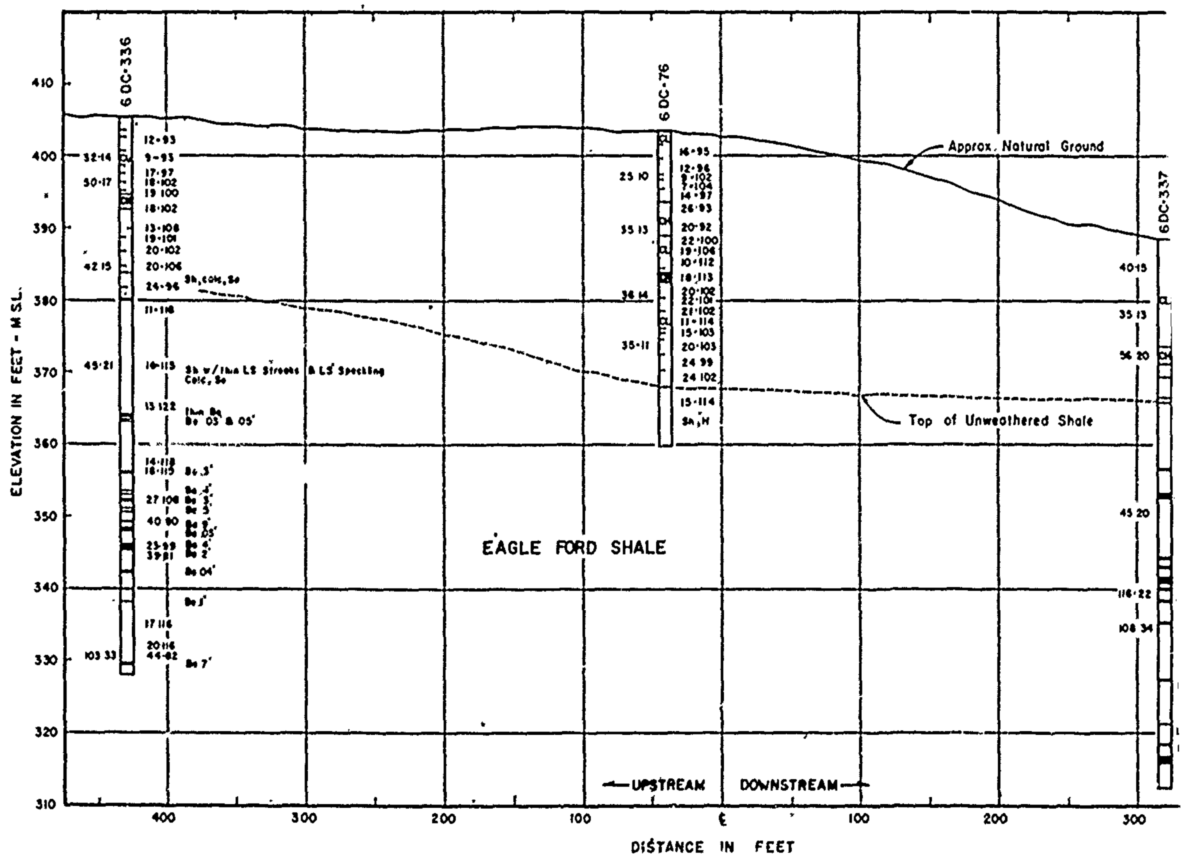
WACO DAM
BOSQUE RIVER, TEXAS

FOUNDATION SECTIONS I

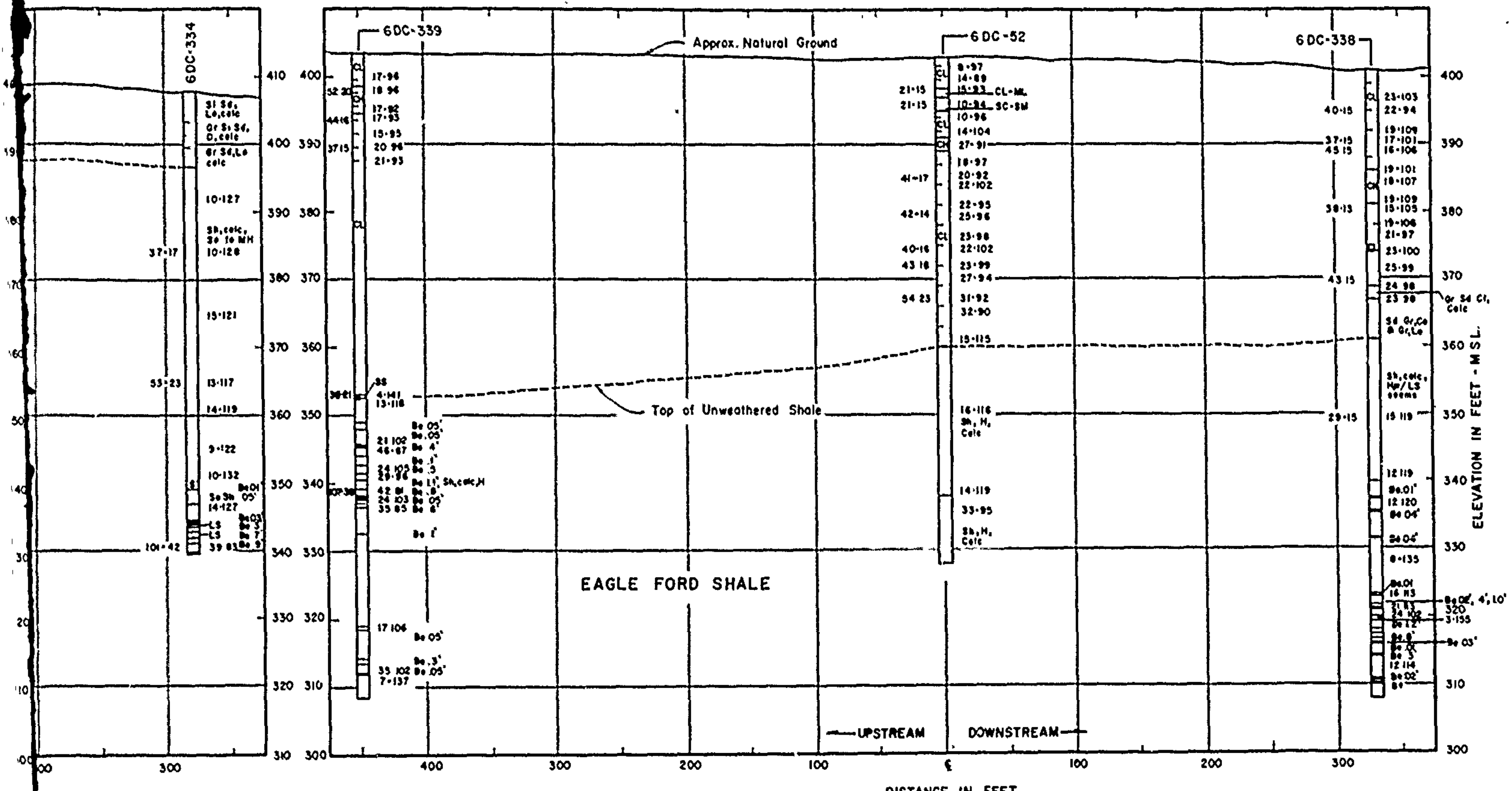
PLATE 20



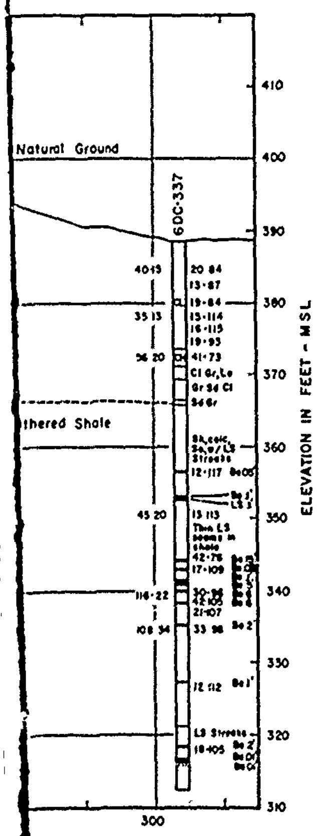
SECTION AT STATION 22+50



SECTION AT STATION 18+40



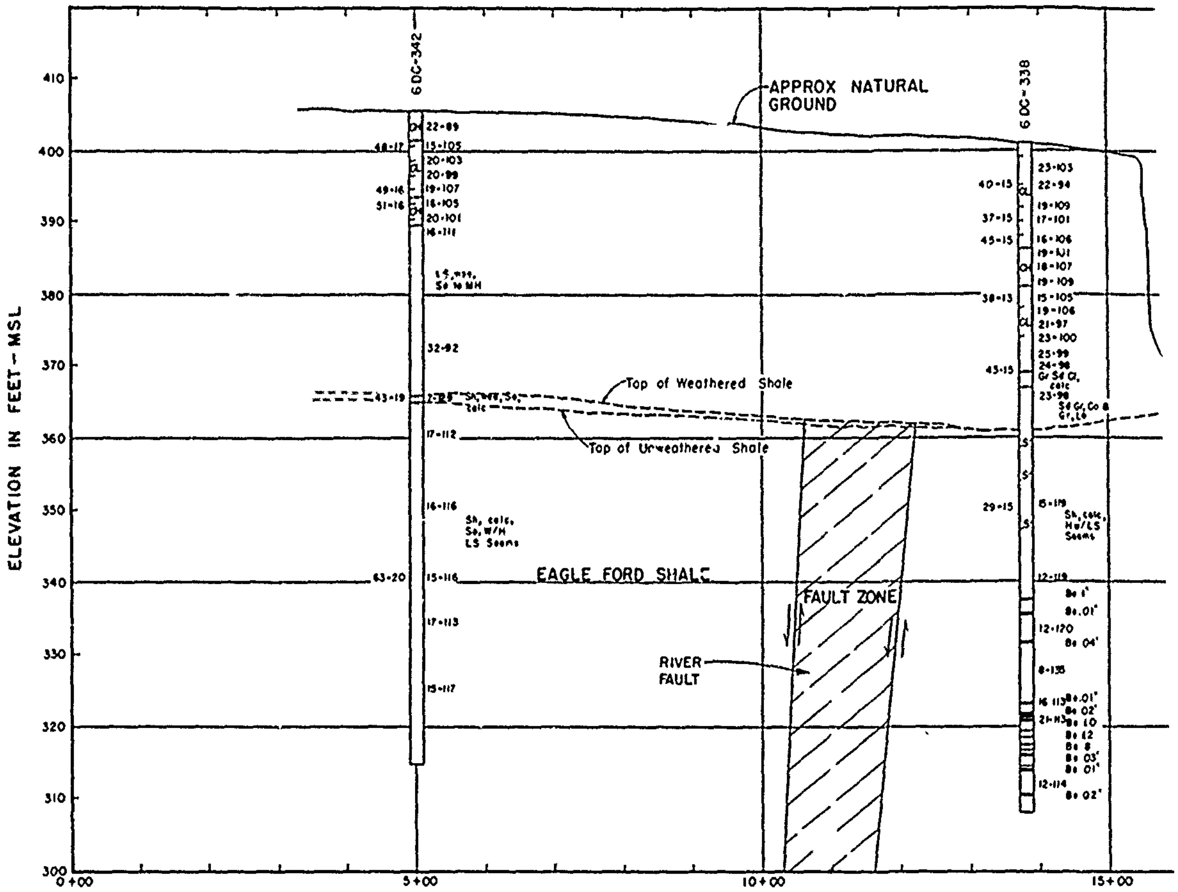
SECTION AT STATION 13+50



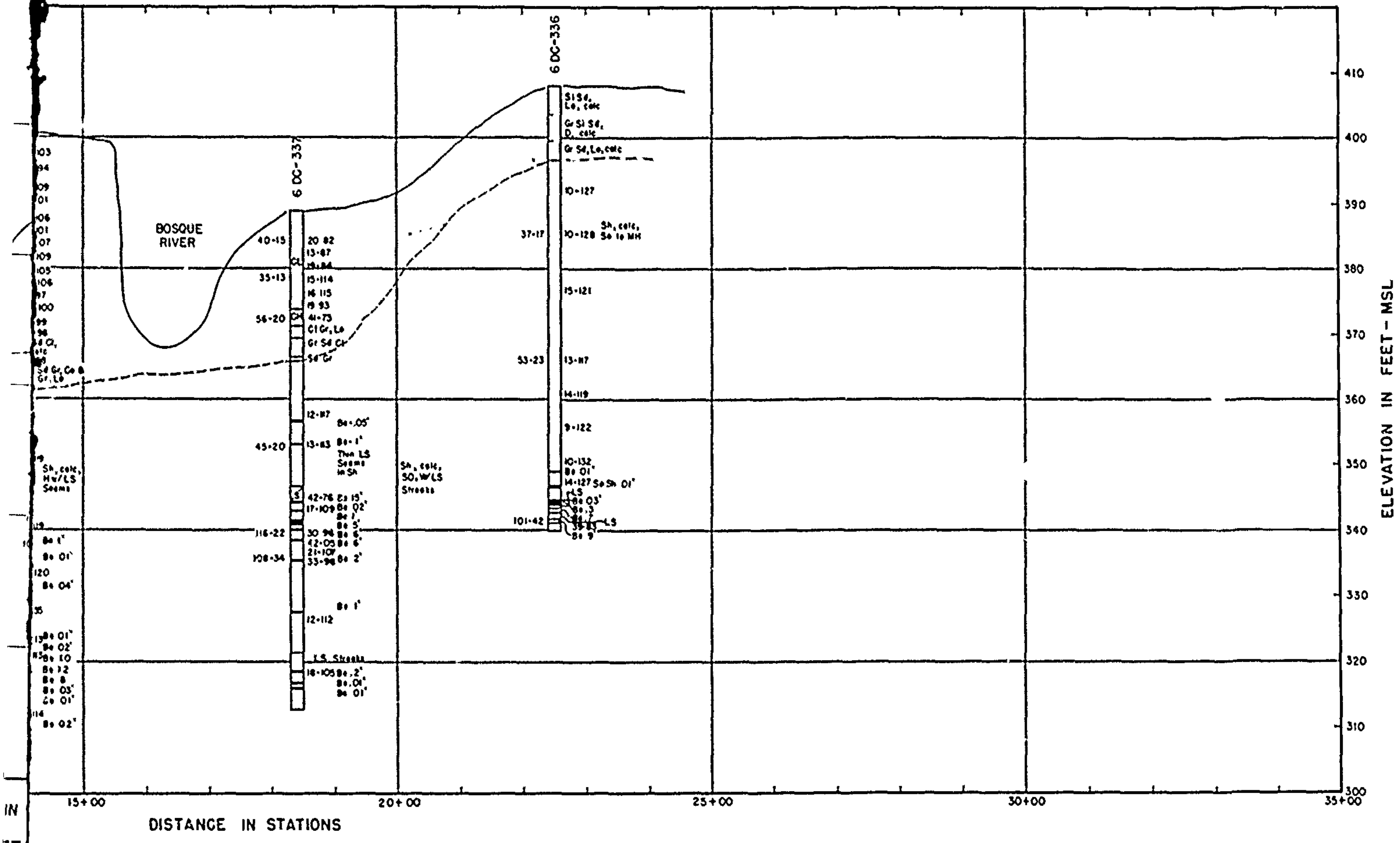
WACO DAM
BOSQUE RIVER, TEXAS

FOUNDATION SECTIONS II

CORPS OF ENGINEERS



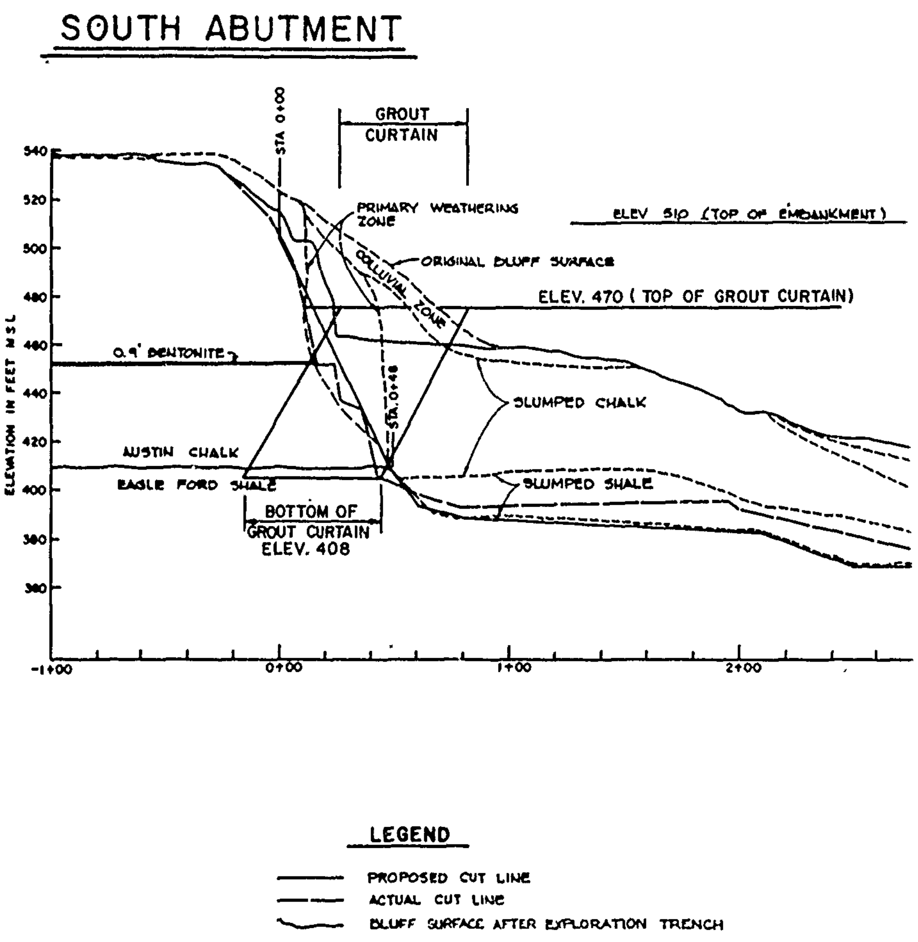
SECTIO



SECTION 320 FEET DOWNSTREAM

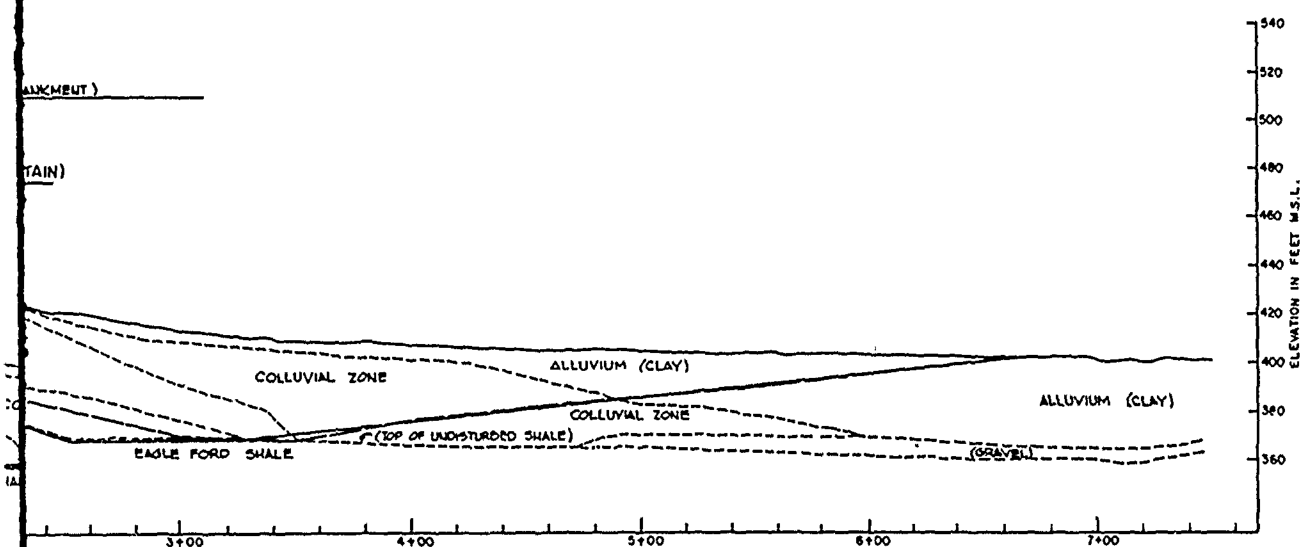
WACO DAM
BOSQUE RIVER, TEXAS

FOUNDATION SECTIONS III



SOUTH ABUTMENT
GROUT CURTAIN
STA 0+00
PRIMARY WEATHERING ZONE
ELEV 510 (TOP OF EMBANKMENT)
ORIGINAL BLUFF SURFACE
COLLUVIAL ZONE
ELEV. 470 (TOP OF GROUT CURTAIN)
0.9' BENTONITE
STA. 0+48
SLUMPED CHALK
AUSTIN CHALK
EAGLE FORD SHALE
SLUMPED SHALE
BOTTOM OF GROUT CURTAIN ELEV. 408
ELEVATION IN FEET M S L
540
520
500
480
460
440
420
400
380
-1+00
0+00
1+00
2+00
LEGEND
PROPOSED CUT LINE
ACTUAL CUT LINE
BLUFF SURFACE AFTER EXPLORATION TRENCH

US ARMY



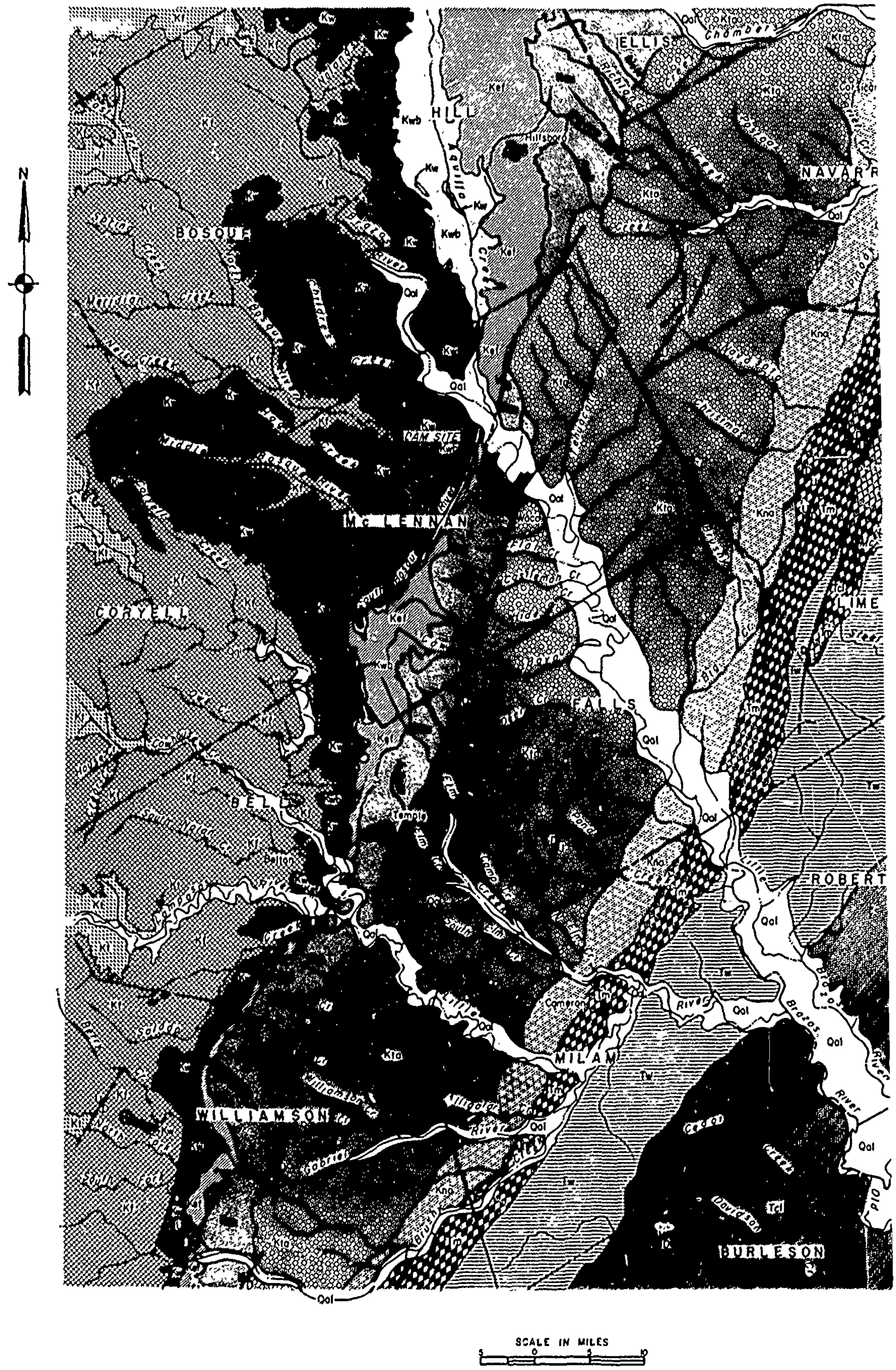
PROFILE

NOTE:
Grout curtain constructed after embankment had been completed to El 470.

WACO DAM
BOSQUE RIVER, TEXAS

SECTION THRU RIGHT ABUTMENT

PLATE 23


ELLIS
HILL
NAVARRO
BOSQUE
DAM SITE
McLENNAN
CORYELL
LIMESTONE
FALLS
BELL
Temple
Belton
ROBERTSON
Cameron
MILAM
WILLIAMSON
BURLESON
SCALE IN MILES

LEGEND

QUATERNARY — RECENT

- Qal — Alluvium

TERTIARY — EOCENE

- Tj — Jackson Group (Undifferentiated)
- Tcl — Claiborne Group
  Includes Yegua Formation, Cook Mountain Formation, Sparta Sand, Weches Greensand Member, Queen City Sand, Reklaw Member, and Carrizo Sand.

UNCONFORMITY

- Tw — Wilcox Group (Undifferentiated)
- Tm — Midway Group
  Includes Wills Point Clay and Kincaid Formation.

UNCONFORMITY

CRETACEOUS — UPPER — GULF SERIES

- Kng — Navarro Group
  Includes Kemp Clay, Corsicana Marl, Nacatoch Sand, and Neylandville Marl.

UNCONFORMITY

- Kta — Taylor Group
  Includes Pecan Gap Chalk Member and Wolfe City Sand Member.

UNCONFORMITY

- Kau — Austin Group (Austin Chalk)

UNCONFORMITY

- Kef — Eagle Ford Group
  Includes South Bosque and Lake Waco Formations.

UNCONFORMITY

- Kwb — Woodbine Group
  Includes Pepper Shale

UNCONFORMITY

CRETACEOUS — LOWER — COMANCHE SERIES

- Kw — Washita Group
  Includes Buda Limestone, Del Rio Clay and Georgetown Limestone.
- Kf — Fredricksburg Group
  Includes Edwards Limestone, Comanche Peak Limestone and Walnut Clay
- Kt — Trinity Group
  Includes Paluxy Sand, Glen Rose Formation and Travis Peak Formation (Trinity Sand)

Geologic Contact

U D — Fault, known and inferred

NOTE:
Geologic information taken from U.S.G.S "Geologic Map of Texas," 1937.

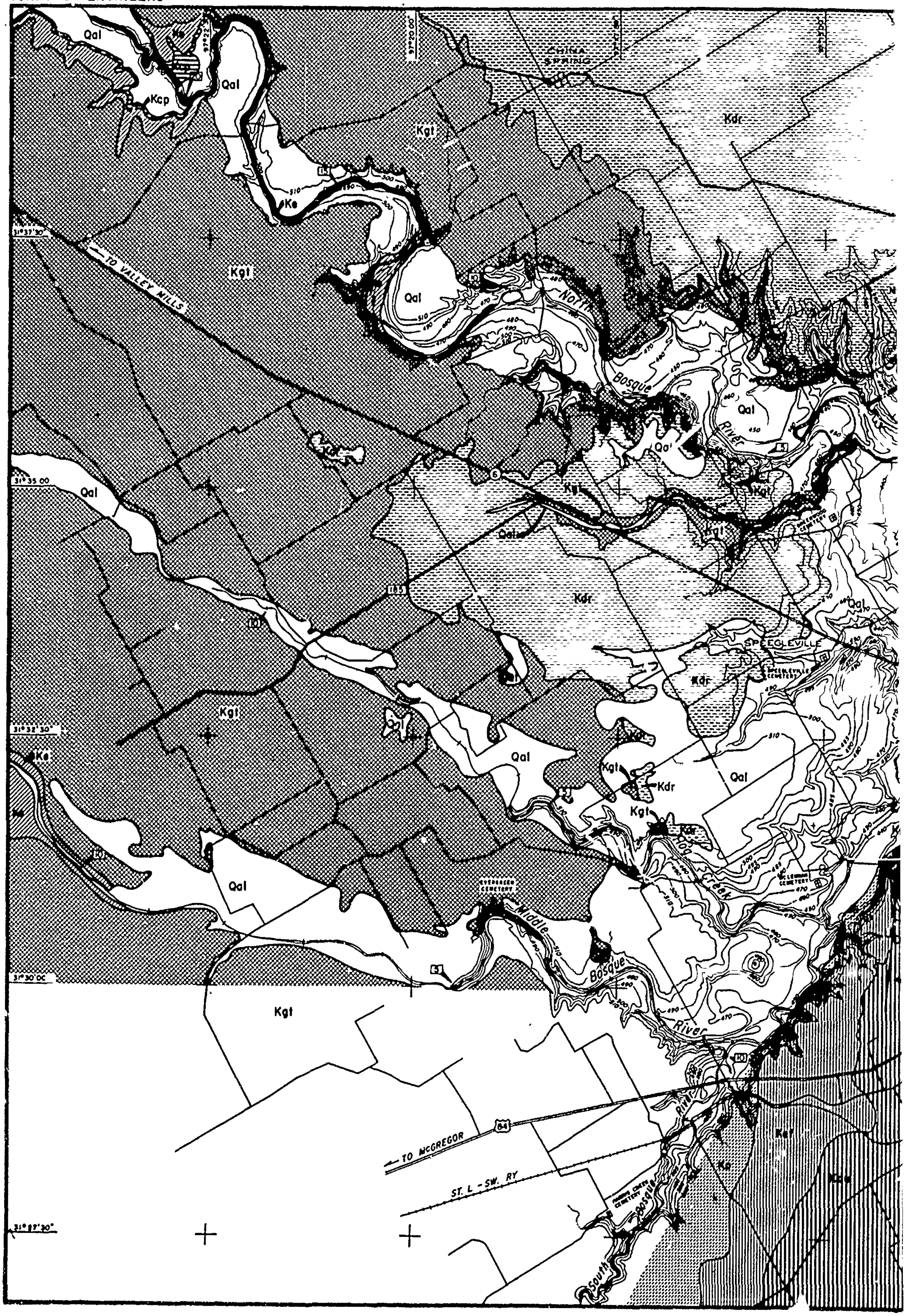
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

REGIONAL GEOLOGY

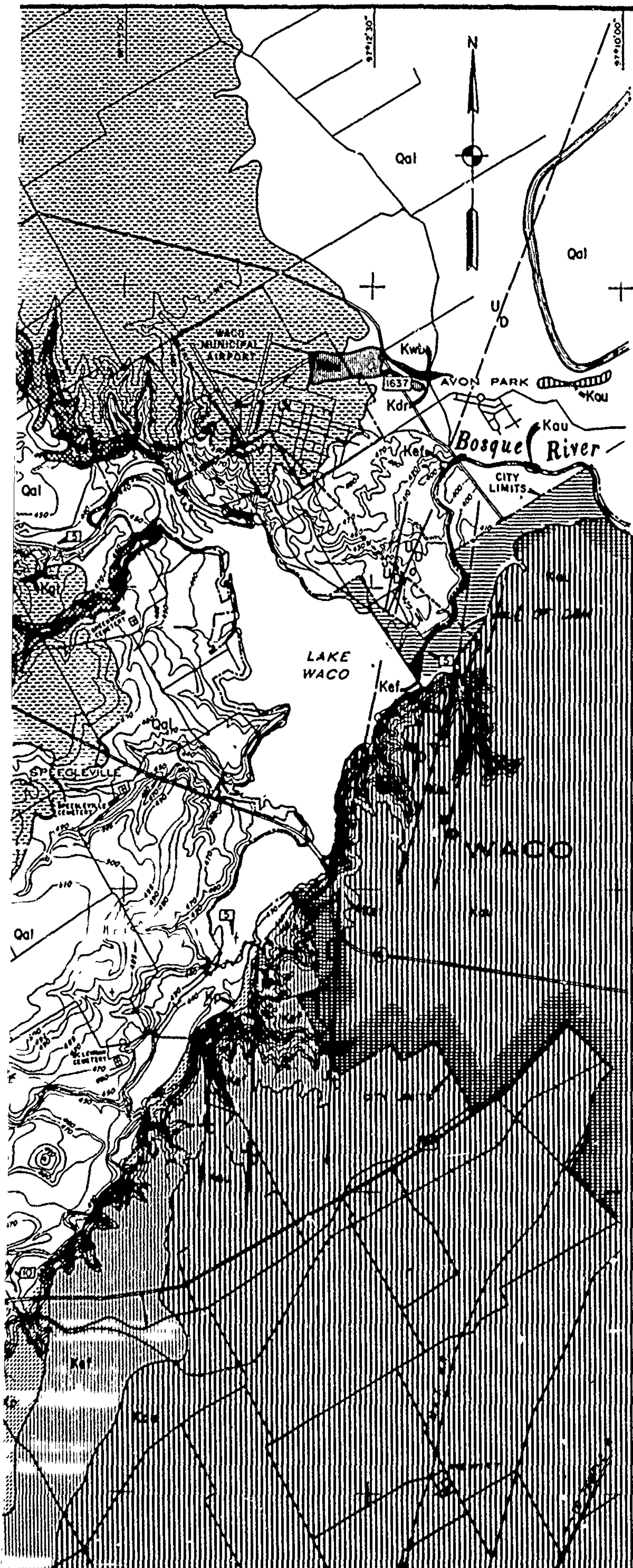
SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN 1953

CORPS OF ENGINEERS



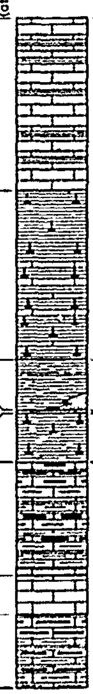
U.S. ARMY



LEGEND

- Qal — ALLUVIAL DEPOSITS, Channel, floodplain and terrace deposits.
- Ka — AUSTIN CHALK
- Kef — EAGLE FORD SHALE
- Kwb — WOODBINE SANDS
- Kp — PEPPER SHALE
- Kbu — BUDA LIMESTONE
- Kdr — DEL RIO SHALE
- Kgt — GEORGETOWN LIMESTONE
- Ke — EDWARDS LIMESTONE
- Kcp — COMANCHE PEAK LIMESTONE
- GEOLOGIC CONTACT
- U/D — FAULT, Known and Inferred

GENERALIZED COLUMNAR SECTION



System	Series	Group	Formation	Thickness (at dam site)	General Description
CRETACEOUS	GULF	AUSTIN	AUSTIN CHALK	238'	Massive, white to yellow chalk beds, with thin dark gray shale interbeds
		EAGLE FORD	EAGLE FORD SHALE	234'	Thinbedded blue gray shales Upper 100'± slightly calcareous Lower 134'± highly calcareous with thin limestone flags and bentonite seams
			PEPPER SHALE / WOODBINE SAND	70'±	Fissile, dark gray, waxy, non-calcareous shale, interfingered with tan gray clayey sands and sandstones
			BUDA LIMESTONE	2'	Thin, hard, yellow, nodular limestone
			DEL RIO SHALE	70'±	Massive, dark green gray calcareous shale, containing occasional thin limestone beds
	COMANCHEAN	WASHITA	GEORGETOWN LIMESTONE	158'	Medium bedded, gray, argillaceous limestone with calcareous shale interbeds
		FREDRICKSBURG	EDWARDS LIMESTONE	55'	Massive to medium bedded, hard, gray to buff limestone.
			COMANCHE PEAK LIMESTONE	102'	Medium bedded, gray, nodular limestone with calcareous shale interbeds

Note Geologic data compiled from unpublished Baylor University Geologic Quadrangle sheets

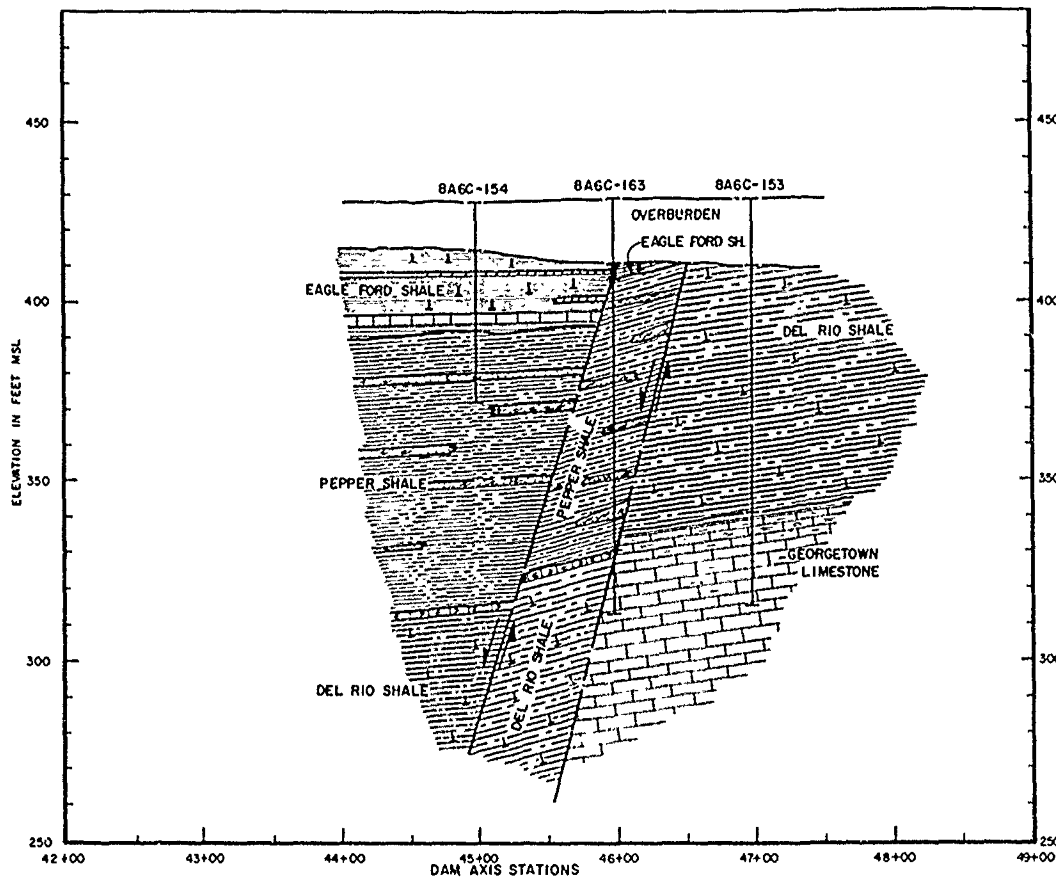
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

RESERVOIR GEOLOGY

SCALE OF FEET
3000 0 3000 6000

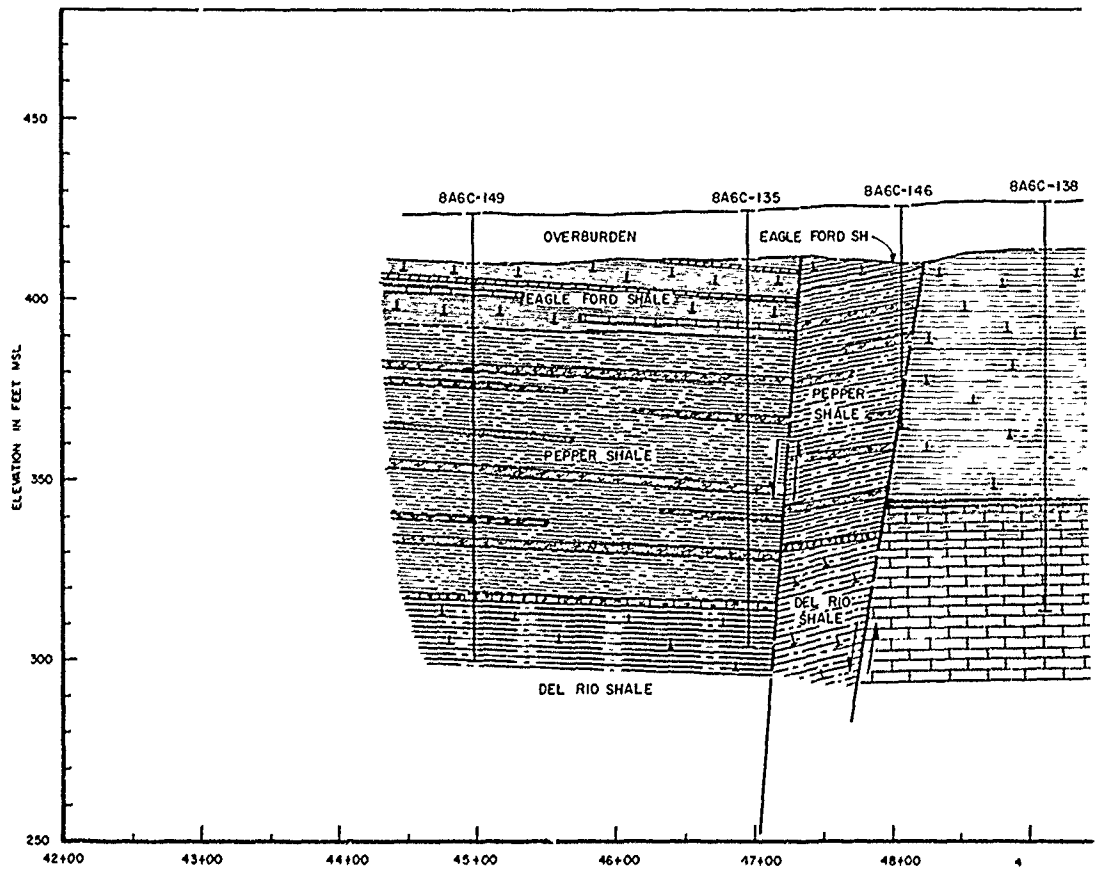
U.S. ARMY ENGINEER DISTRICT, FORT WORTH JAN 1963

CORPS OF ENGINEERS

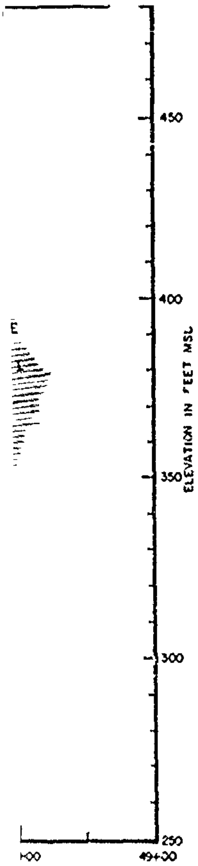


SECTION A-A

2450' UPSTREAM FROM CENTERLINE



2000' U



LEGEND

AUSTIN CHALK
TAN-GRAY AND LIGHT GRAY MARL - CHALK

EAGLE FORD SHALE
SOUTH BOSQUE FORMATION SLIGHTLY CALCAREOUS, DARK GRAY TO BLACK COMPACTION SHALE
LAKE WACO FORMATION HIGHLY CALCAREOUS, DARK GRAY TO BLACK CEMENTATION SHALE WITH BENTONITE AND MARL SEAMS

PEPPER SHALE
NON CALCAREOUS, SOFT, BLACK COMPACTION SHALE WITH SILTSTONE AND SANDY SILTSTONE SEAMS

DEL RIO SHALE
SOFT, CALCAREOUS, GREENISH GRAY COMPACTION SHALE

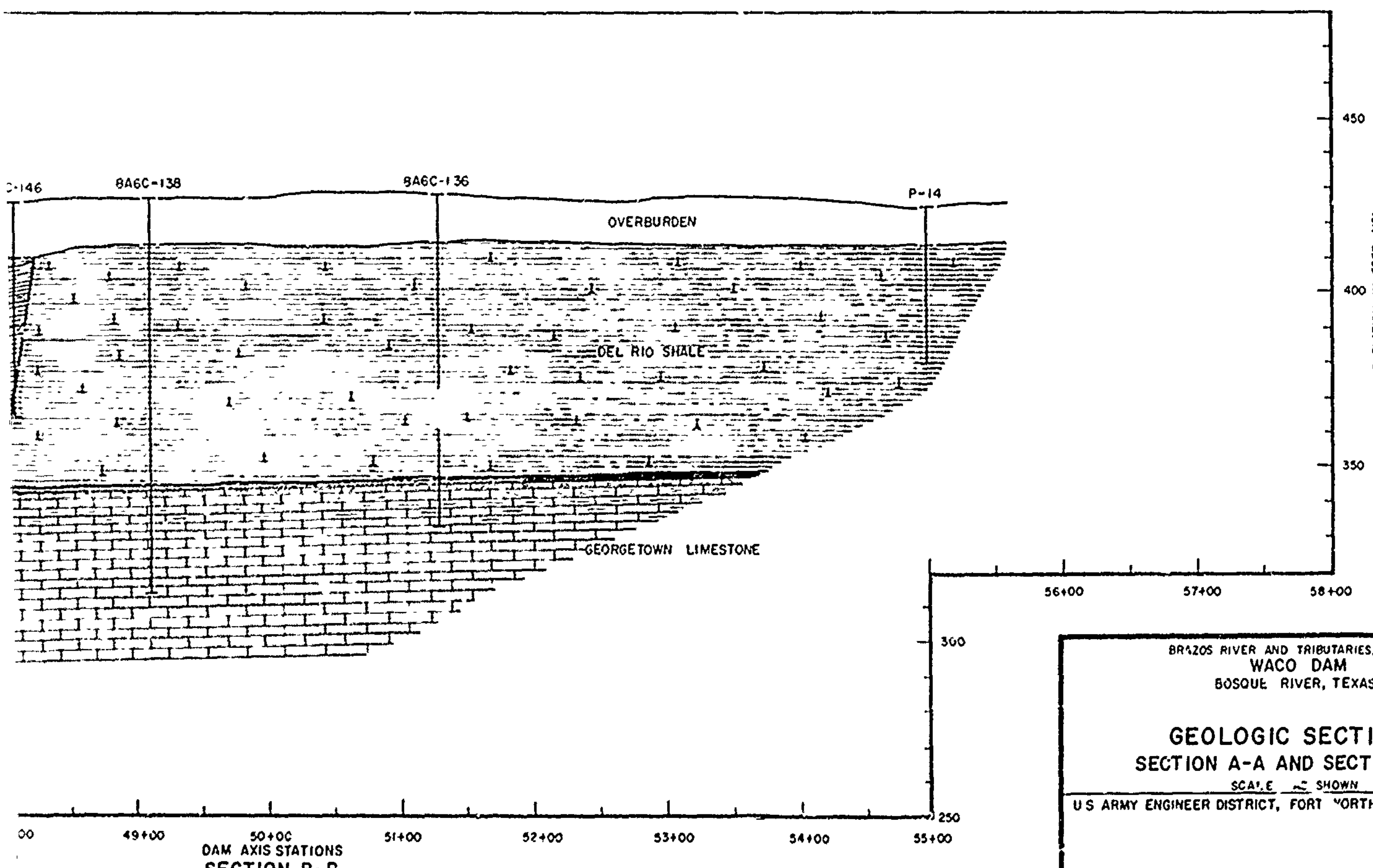
GEORGETOWN LIMESTONE
ARGILLACEOUS, HARD, LIGHT GRAY LIMESTONE

GEOLOGIC CONTACT

FAULT

LOCATION OF BORING IN SECTION

THICKNESS AND POSITION OF INTRAFORMATIMAL FEATURES, SUCH AS LIMESTONE INTERBEDS, BENTONITE SEAMS, AND SILTSTONE SEAMS IS DIAGRAMTIC ONLY EXCEPTIONS TO THIS ARE THE PRESENCE OF THE FOSSILIFEROUS LIMESTONE AT THE TOP OF THE DEL RIO SHALE, THE ABSENCE OF LIMESTONE AND BENTONITE SEAMS IN THE UPPER 101' OF THE EAGLE FORD SHALE, AND THE ABSENCE OF SILTSTONE SEAMS IN THE TOP AND BOTTOM 10' OF THE PEPPER SHALE



SECTION B-B
2000' UPSTREAM FROM CENTERLINE

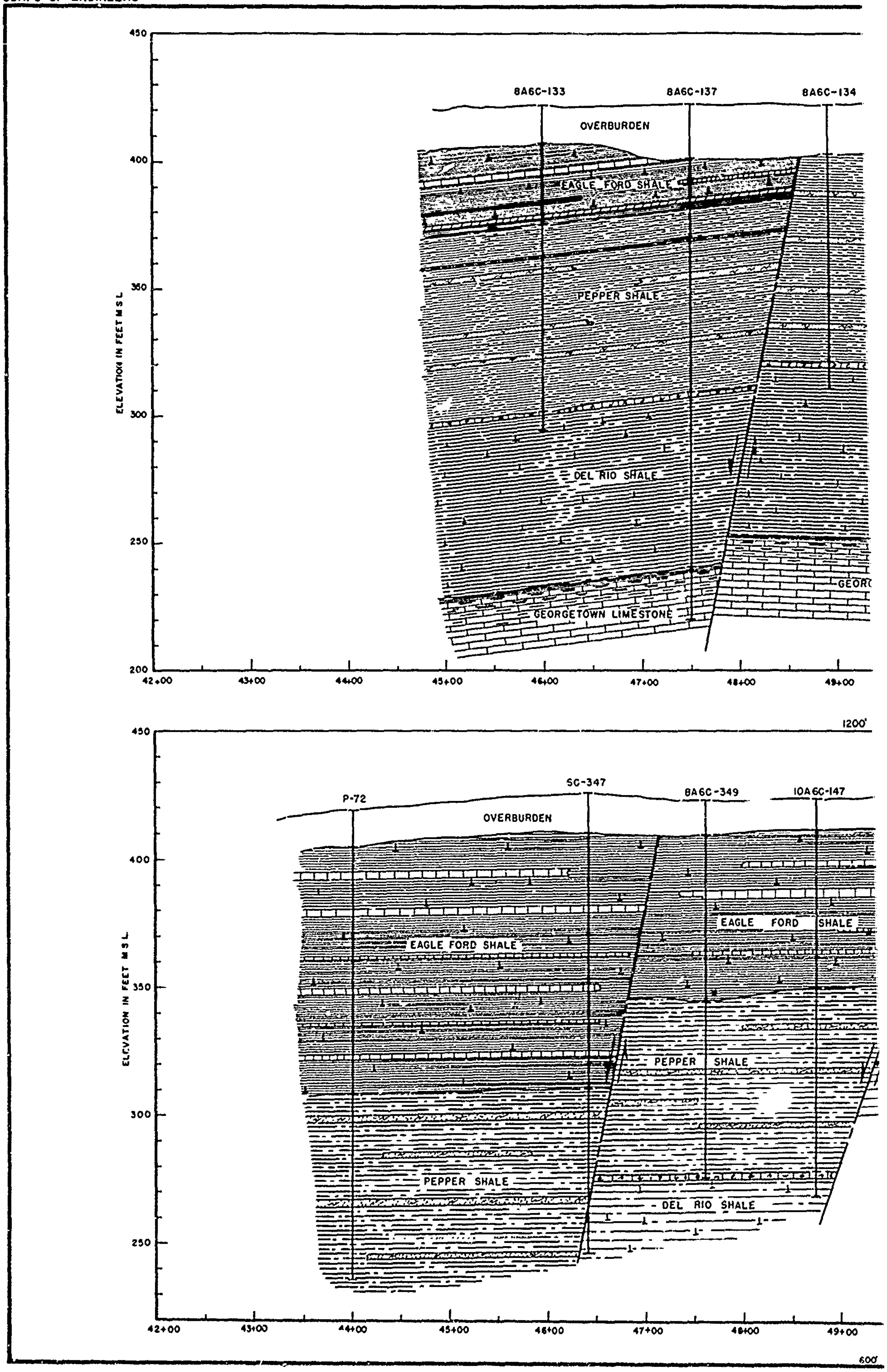
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

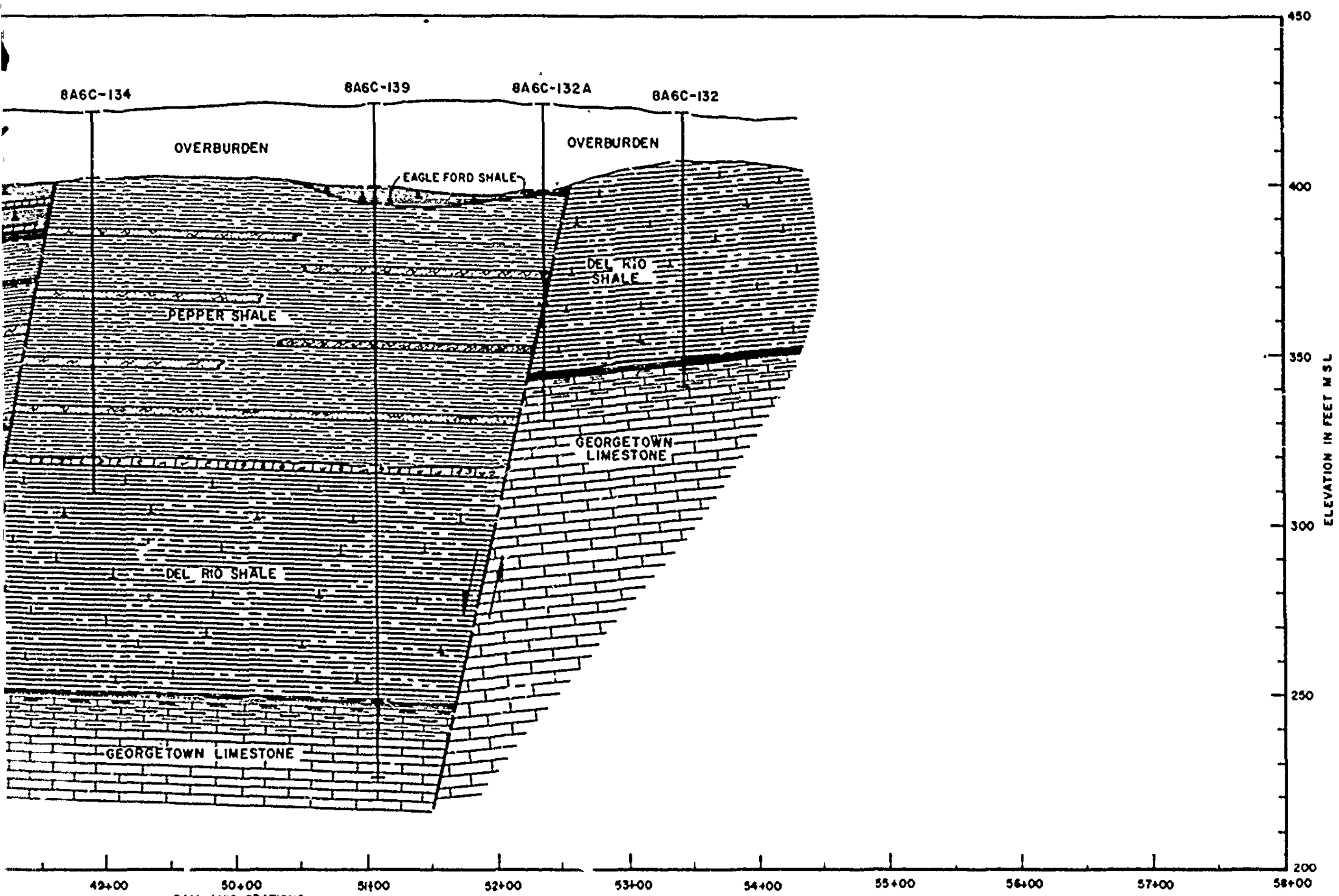
GEOLOGIC SECTIONS
SECTION A-A AND SECTION B-B

SCALE AS SHOWN

U S ARMY ENGINEER DISTRICT, FORT WORTH JAN 1963

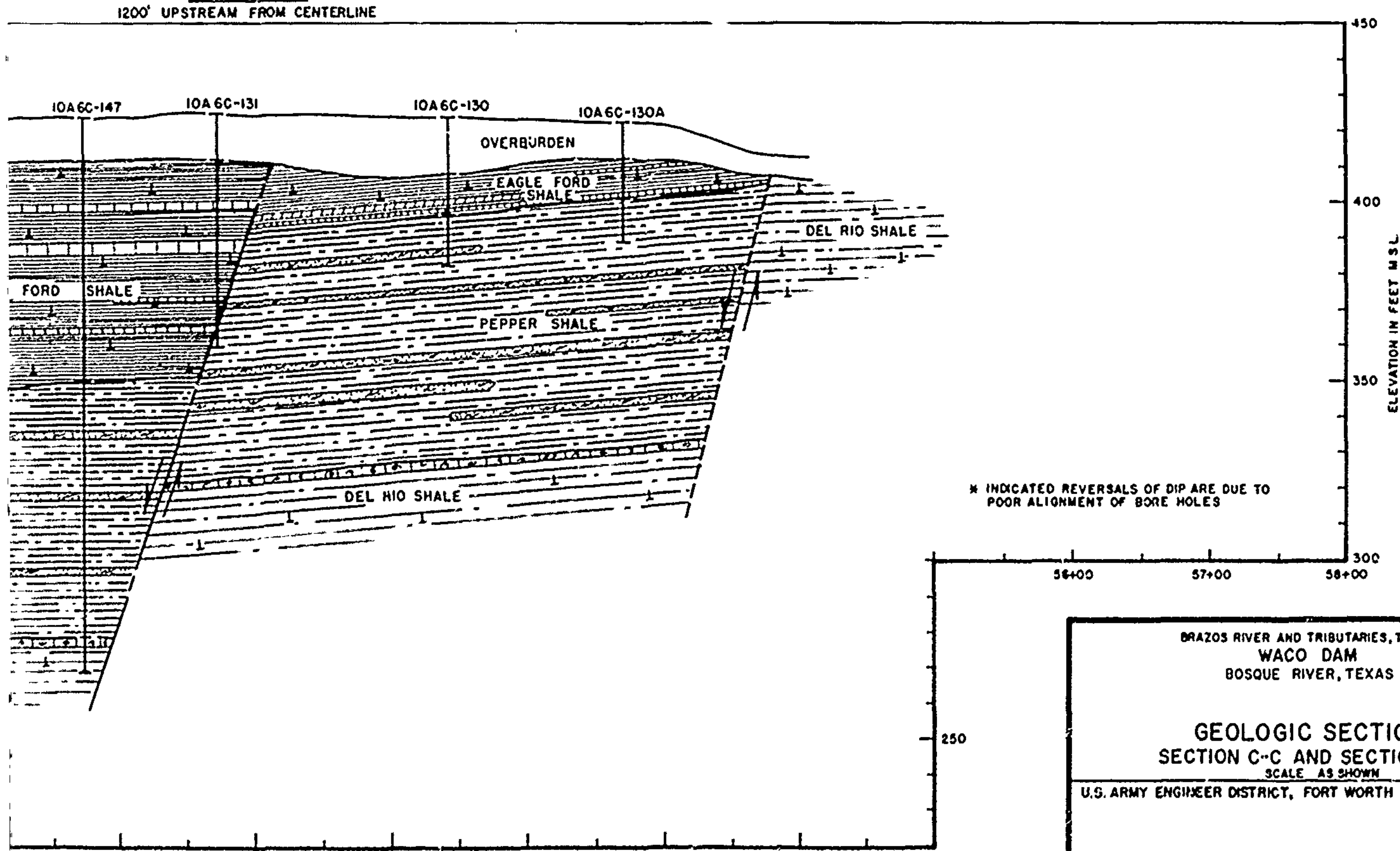
CORPS OF ENGINEERS





SECTION C-C

1200' UPSTREAM FROM CENTERLINE



SECTION D-D

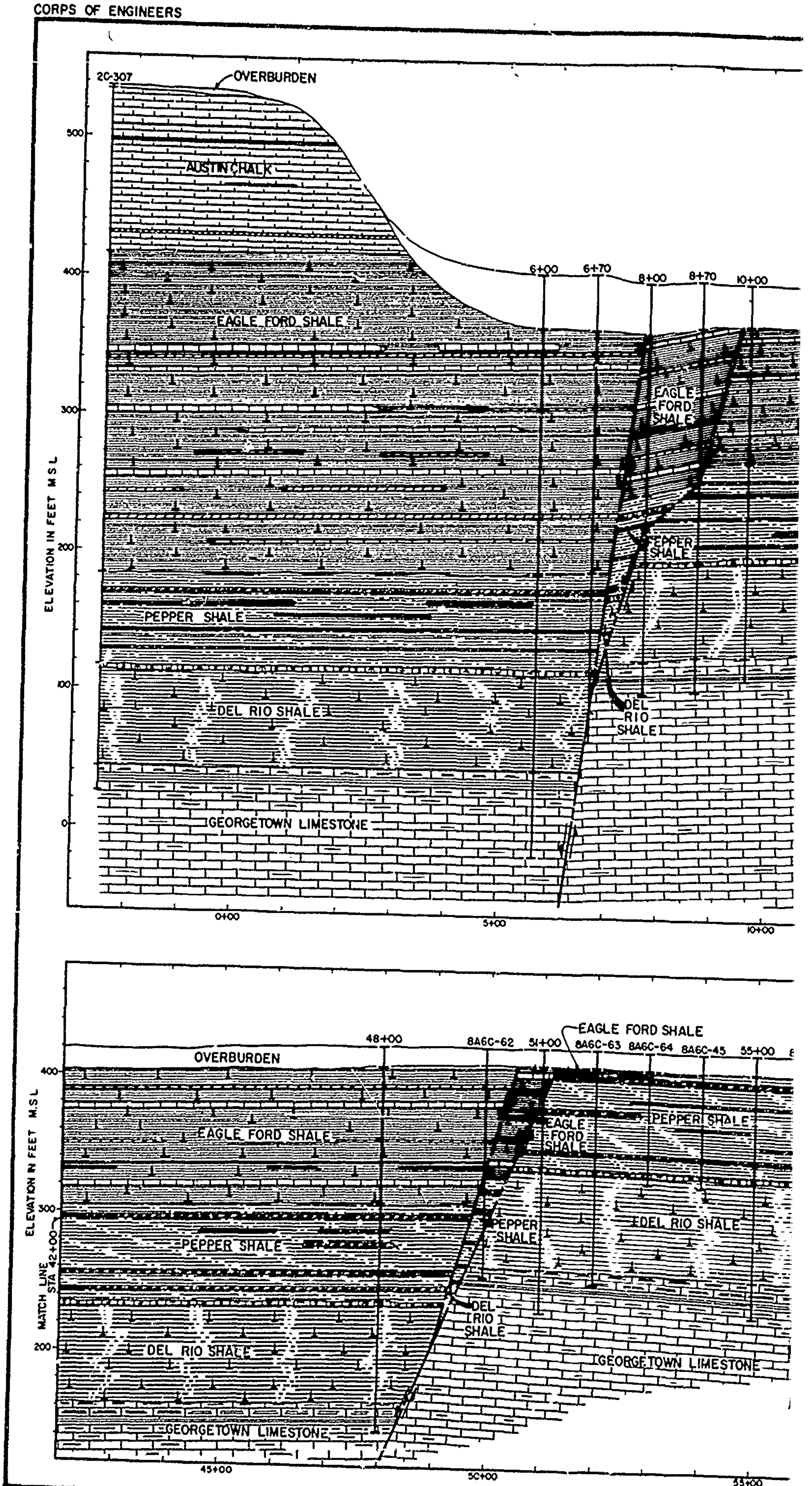
600' TO 900' UPSTREAM FROM CENTERLINE

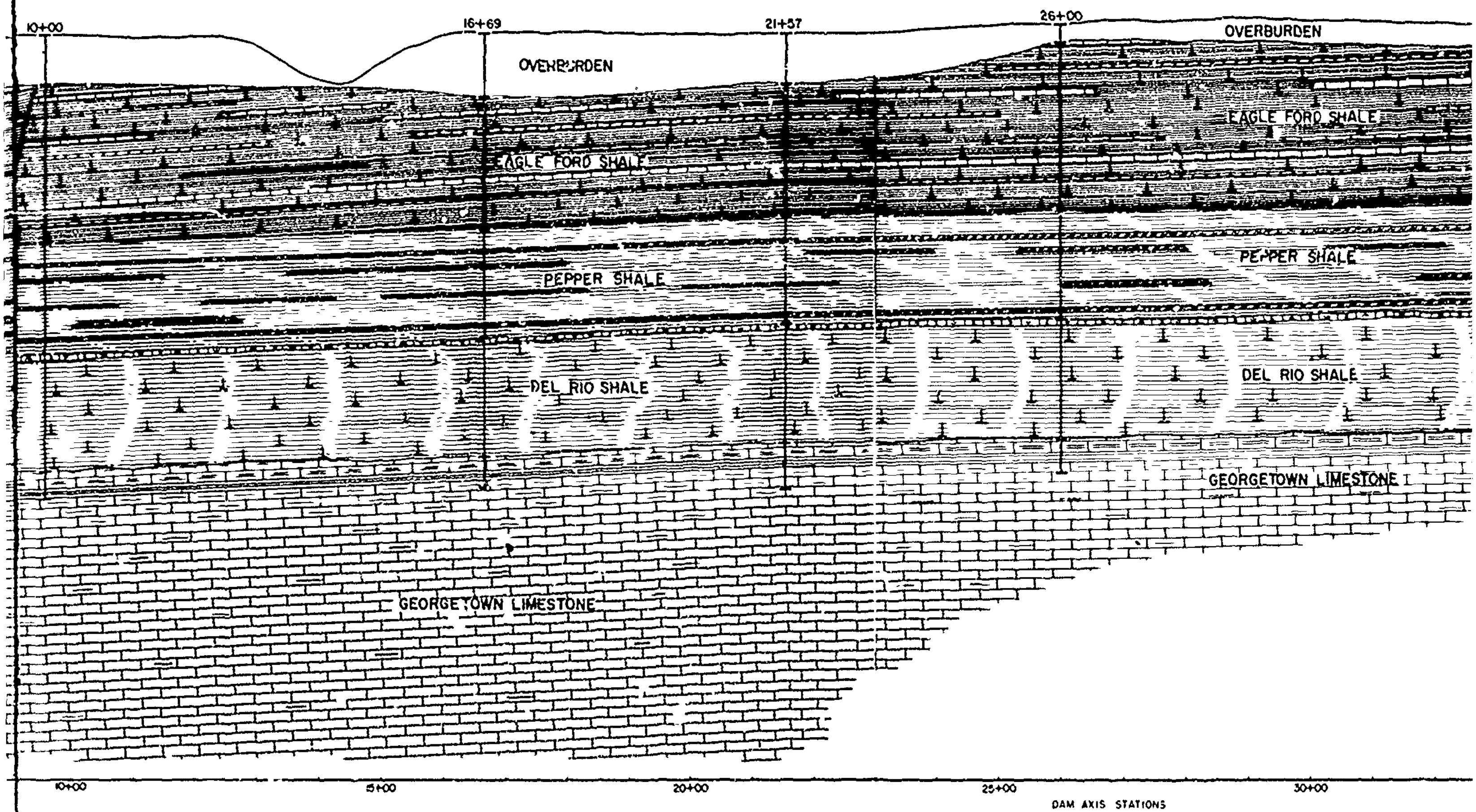
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

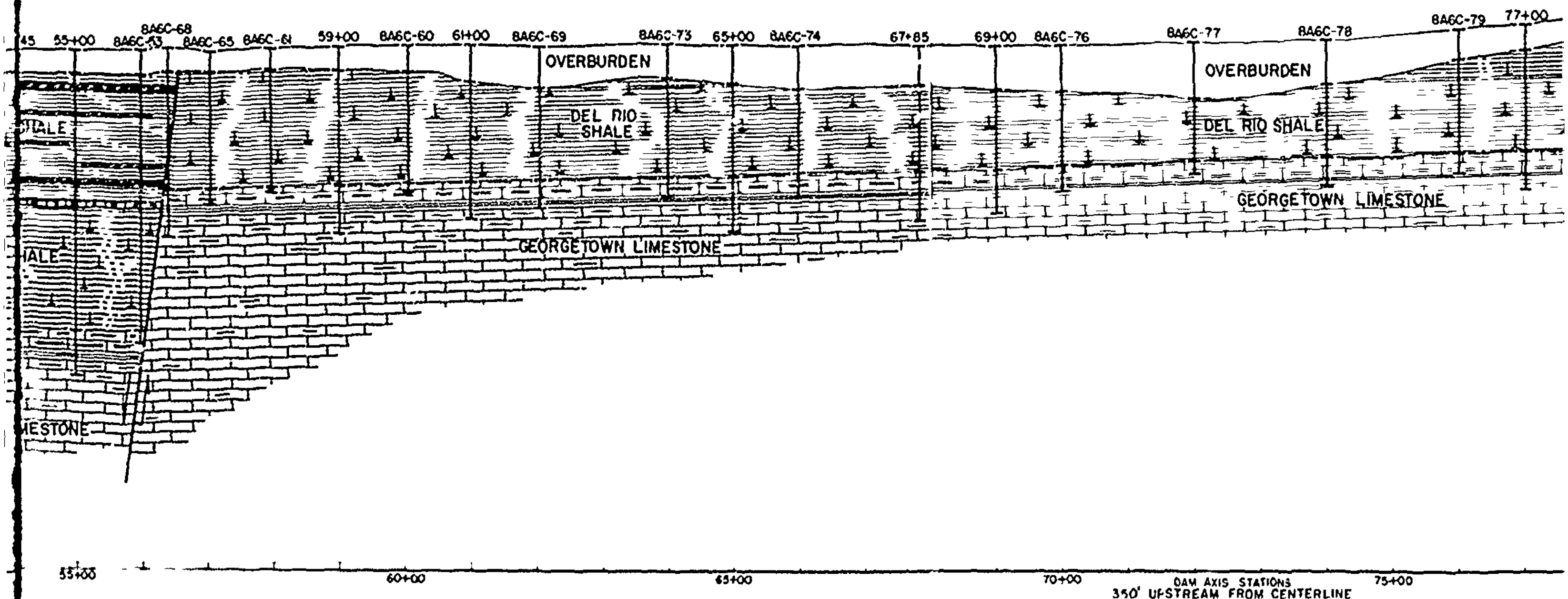
GEOLOGIC SECTIONS
SECTION C-C AND SECTION D-D
SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN 1963

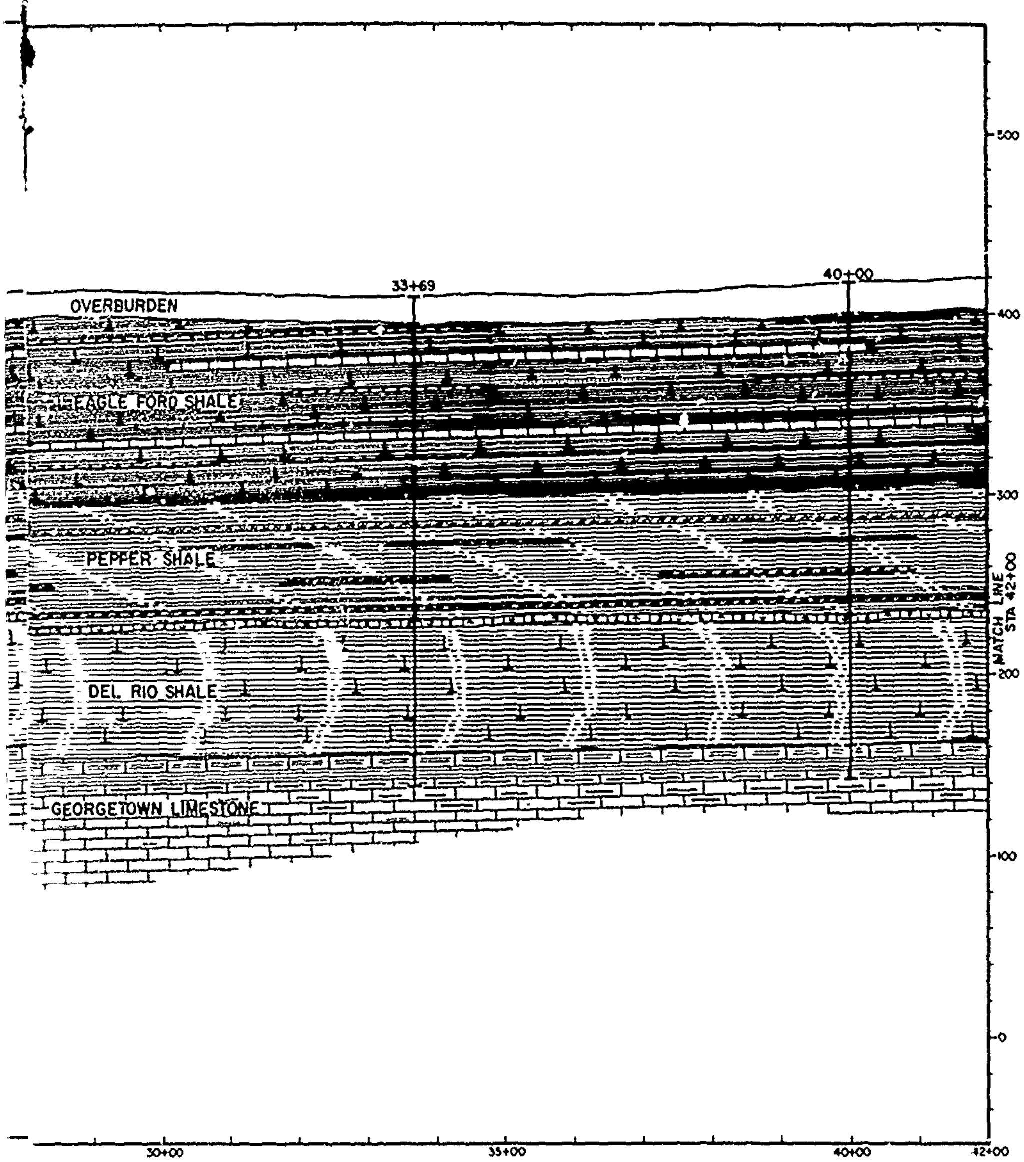
CORPS OF ENGINEERS

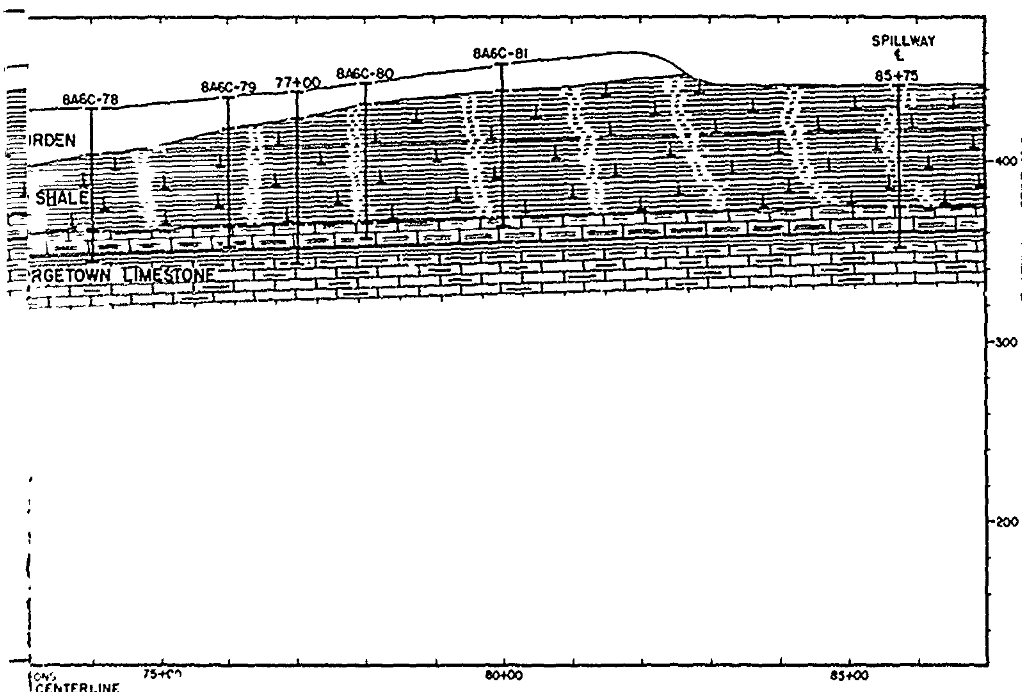



10+00
16+69
21+57
26+00
OVERBURDEN
OVERBURDEN
EAGLE FORD SHALE
EAGLE FORD SHALE
PEPPER SHALE
PEPPER SHALE
DEL RIO SHALE
DEL RIO SHALE
GEORGETOWN LIMESTONE
GEORGETOWN LIMESTONE
10+00
15+00
20+00
25+00
30+00
DAM AXIS STATIONS


55+00
8A6C-68
8A6C-65
8A6C-61
59+00
8A6C-60
61+00
8A6C-69
8A6C-73
65+00
8A6C-74
67+85
69+00
8A6C-76
8A6C-77
8A6C-78
8A6C-79
77+00
OVERBURDEN
OVERBURDEN
DEL RIO SHALE
DEL RIO SHALE
GEORGETOWN LIMESTONE
GEORGETOWN LIMESTONE
55+00
60+00
65+00
70+00
DAM AXIS STATIONS
350' UPSTREAM FROM CENTERLINE
75+00

US ARMY





BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

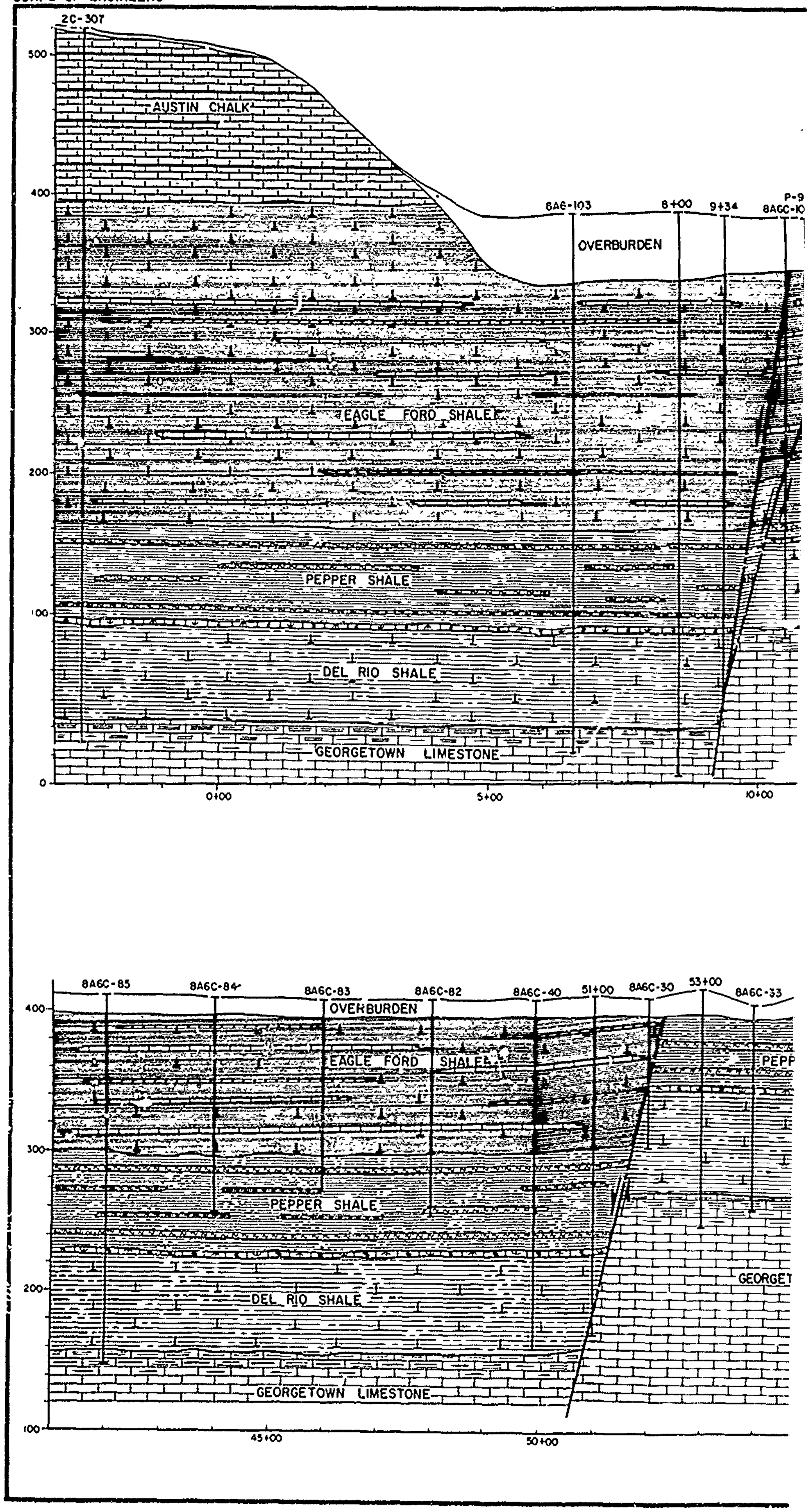
GEOLOGIC SECTION

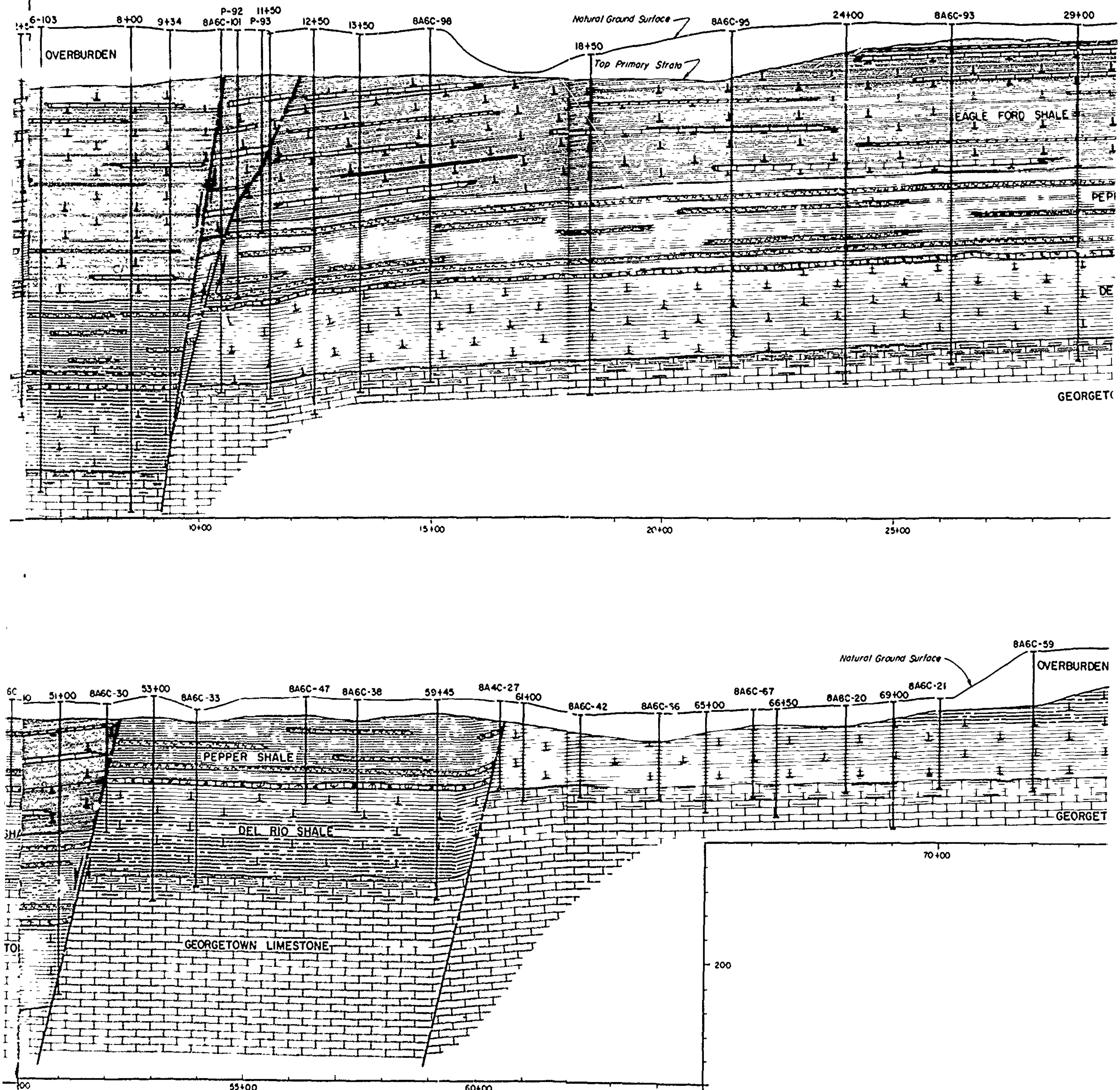
SECTION E-E

IN 2 SHEETS	SCALE AS SHOWN	SHEET NO 2
U S ARMY ENGINEER DISTRICT, FORT WORTH		JAN 1963

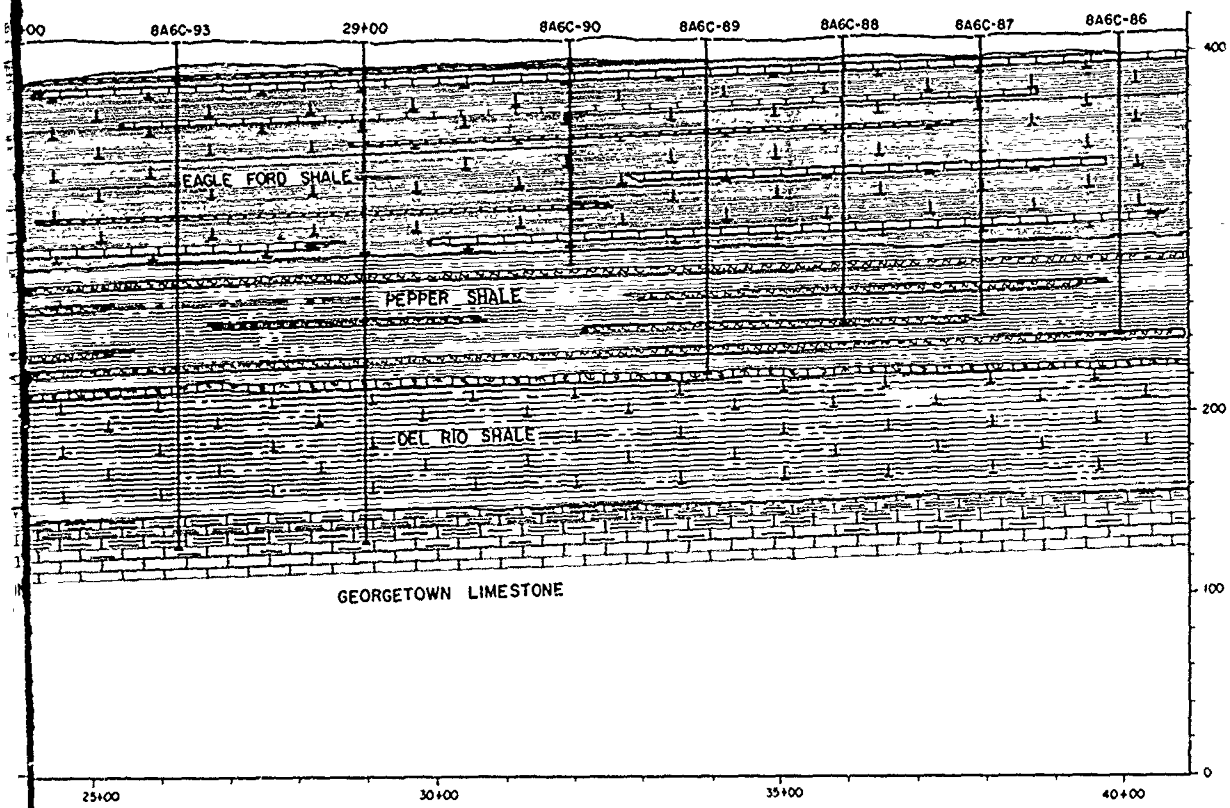
PLATE 28

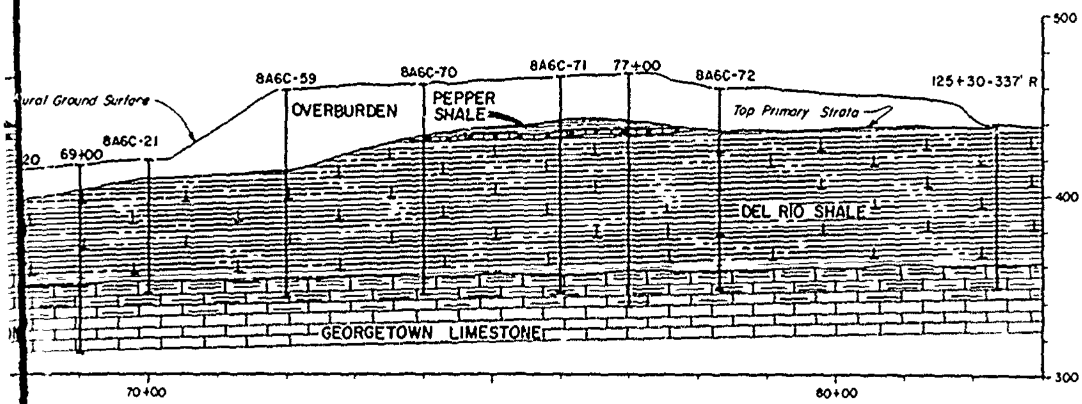
CORPS OF ENGINEERS





DAM AXIS STATIONS
400' DOWNSTREAM FROM CENTERLINE





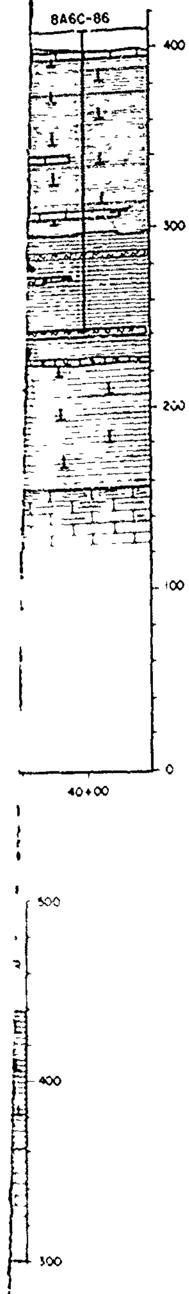
BRAZOS

GE(

IN 2 SHEETS

U.S ARMY ENGIN

U.S. ARMY



BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

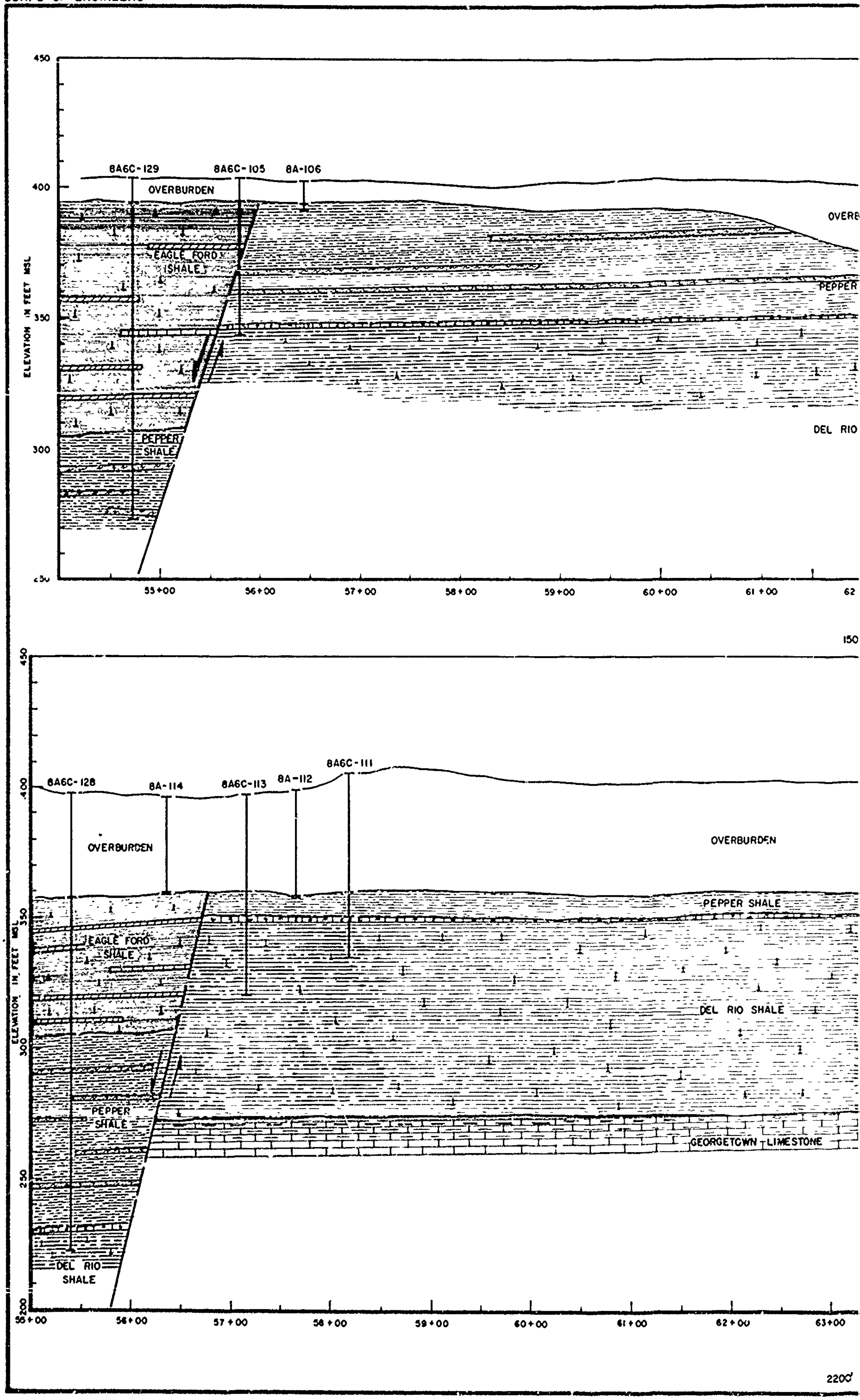
GEOLOGIC SECTION
SECTION F-F

IN 2 SHEETS — SCALE AS SHOWN — SHEET NO. 2

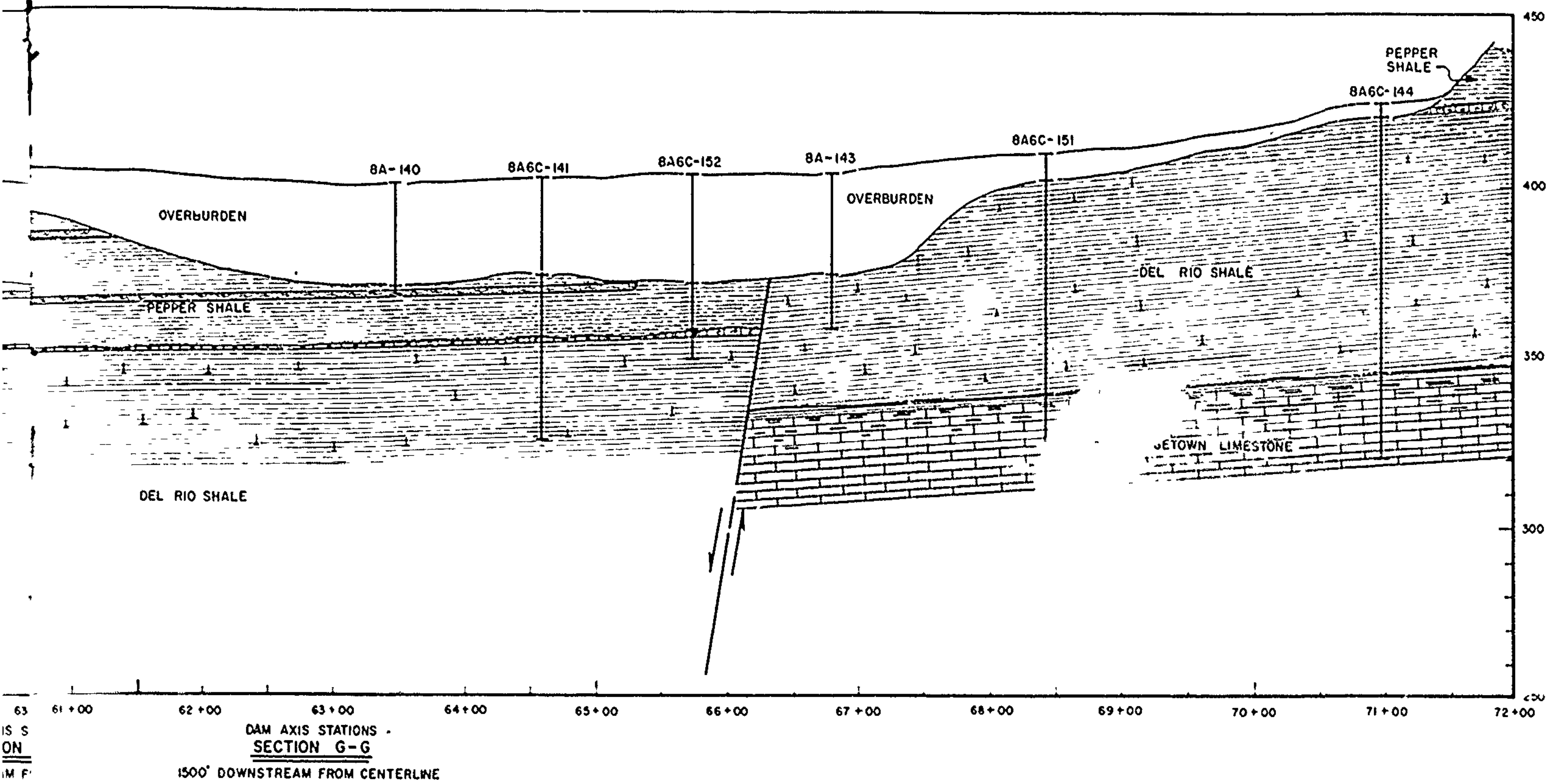
U.S. ARMY ENGINEER DISTRICT, FORT WORTH

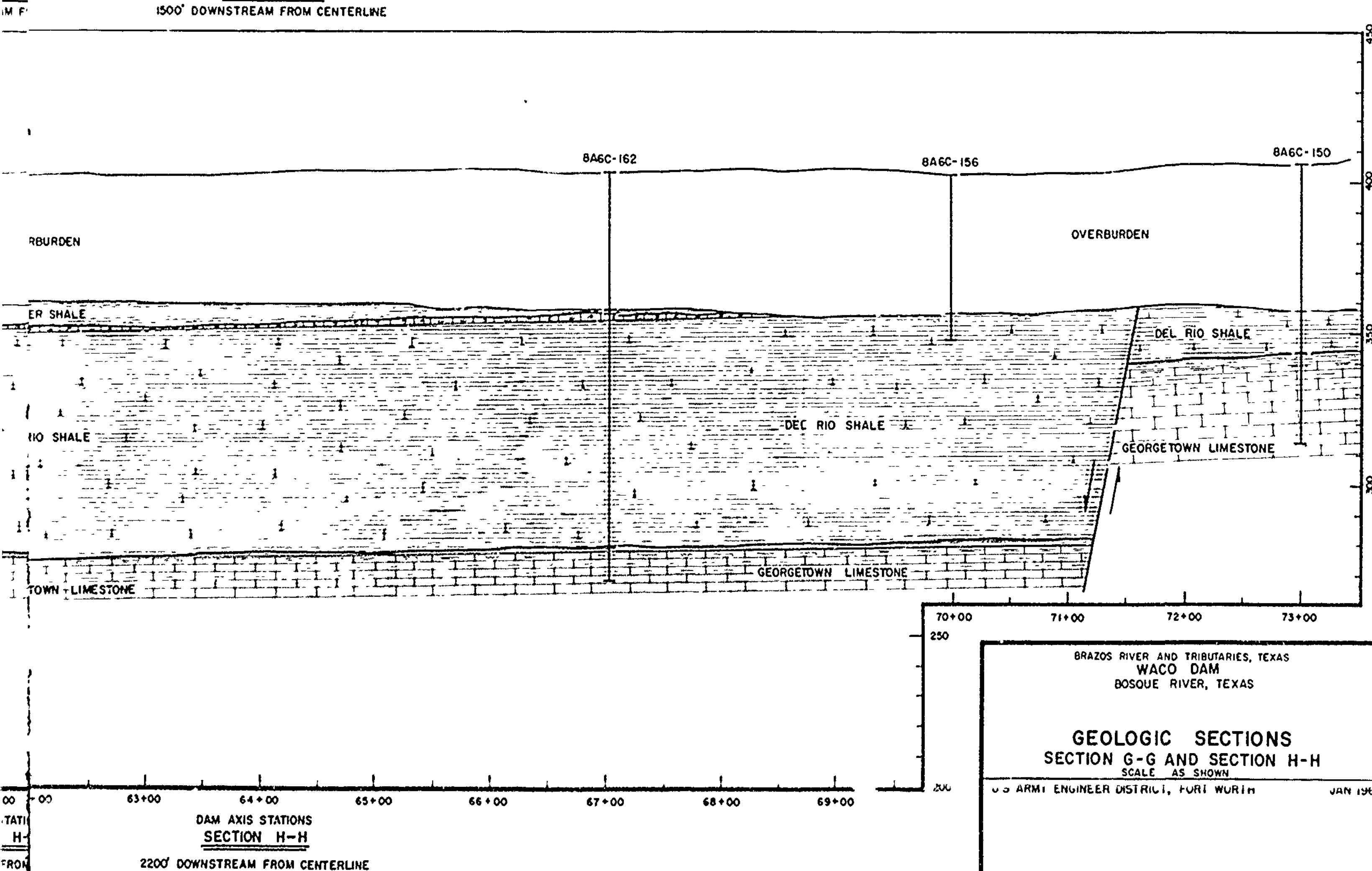
PLATE 29

CORPS OF ENGINEERS



US ARMY





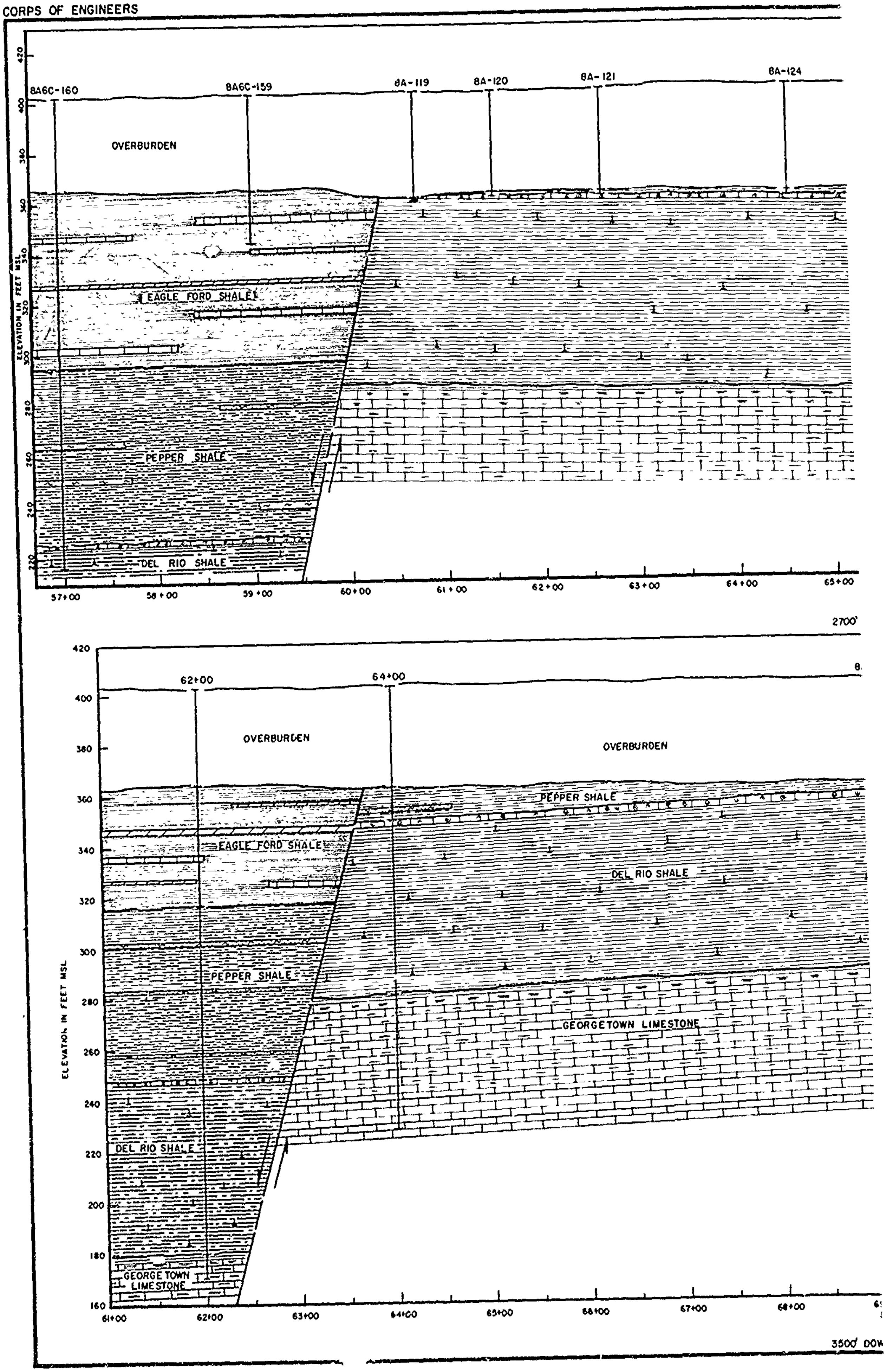
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

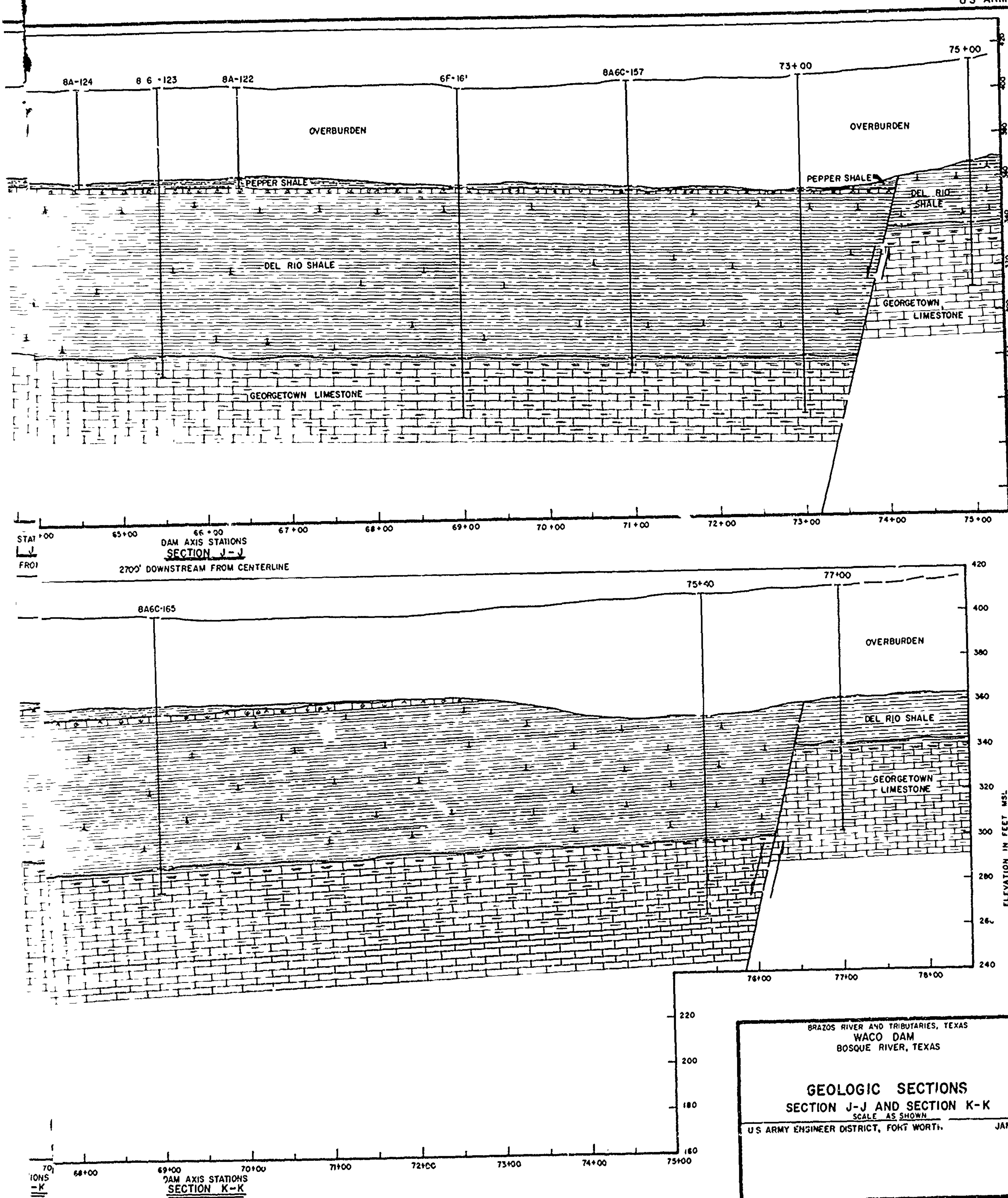
GEOLOGIC SECTIONS
SECTION G-G AND SECTION H-H
SCALE AS SHOWN

US ARMY ENGINEER DISTRICT, FORT WORTH JAN 1963

PLATE 30

CORPS OF ENGINEERS





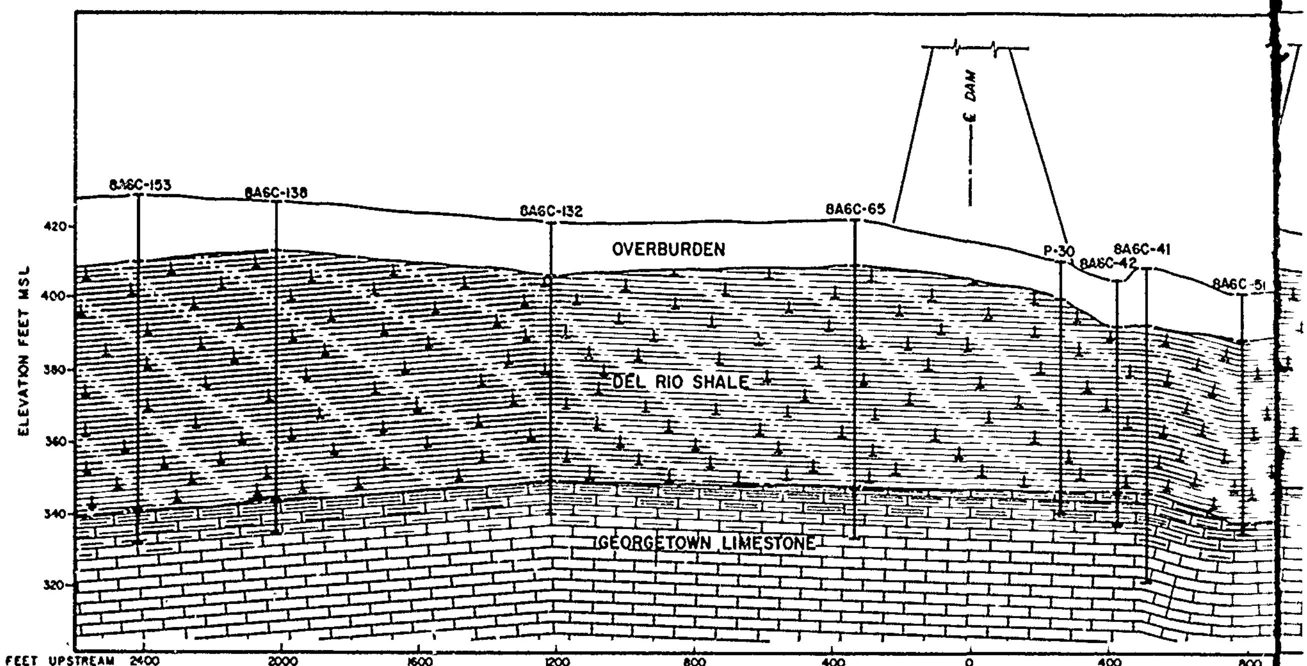
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

GEOLOGIC SECTIONS
SECTION J-J AND SECTION K-K
SCALE AS SHOWN

US ARMY ENGINEER DISTRICT, FORT WORTH — JAN 1963

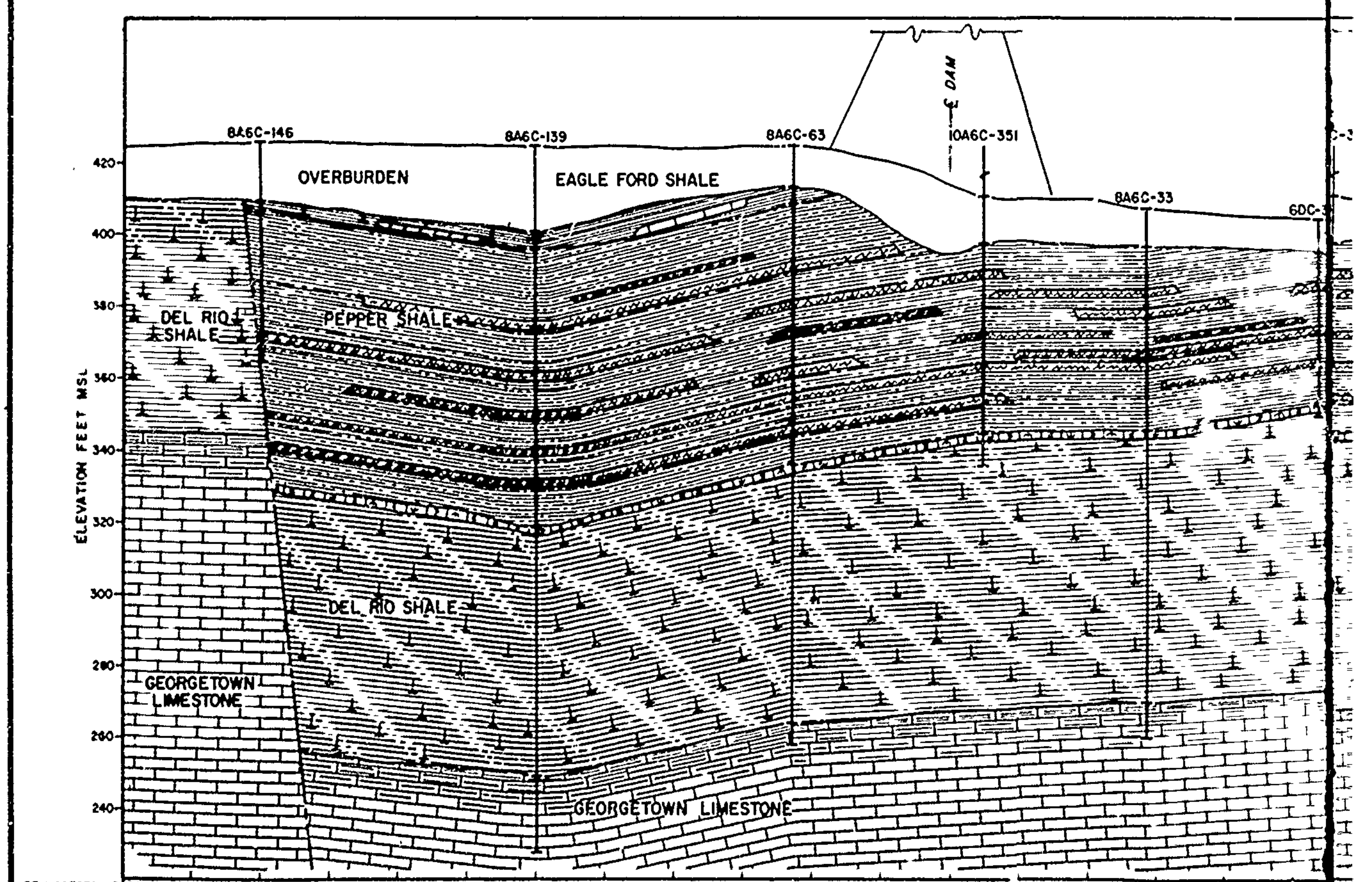
PLATE 31

CORPS OF ENGINEERS



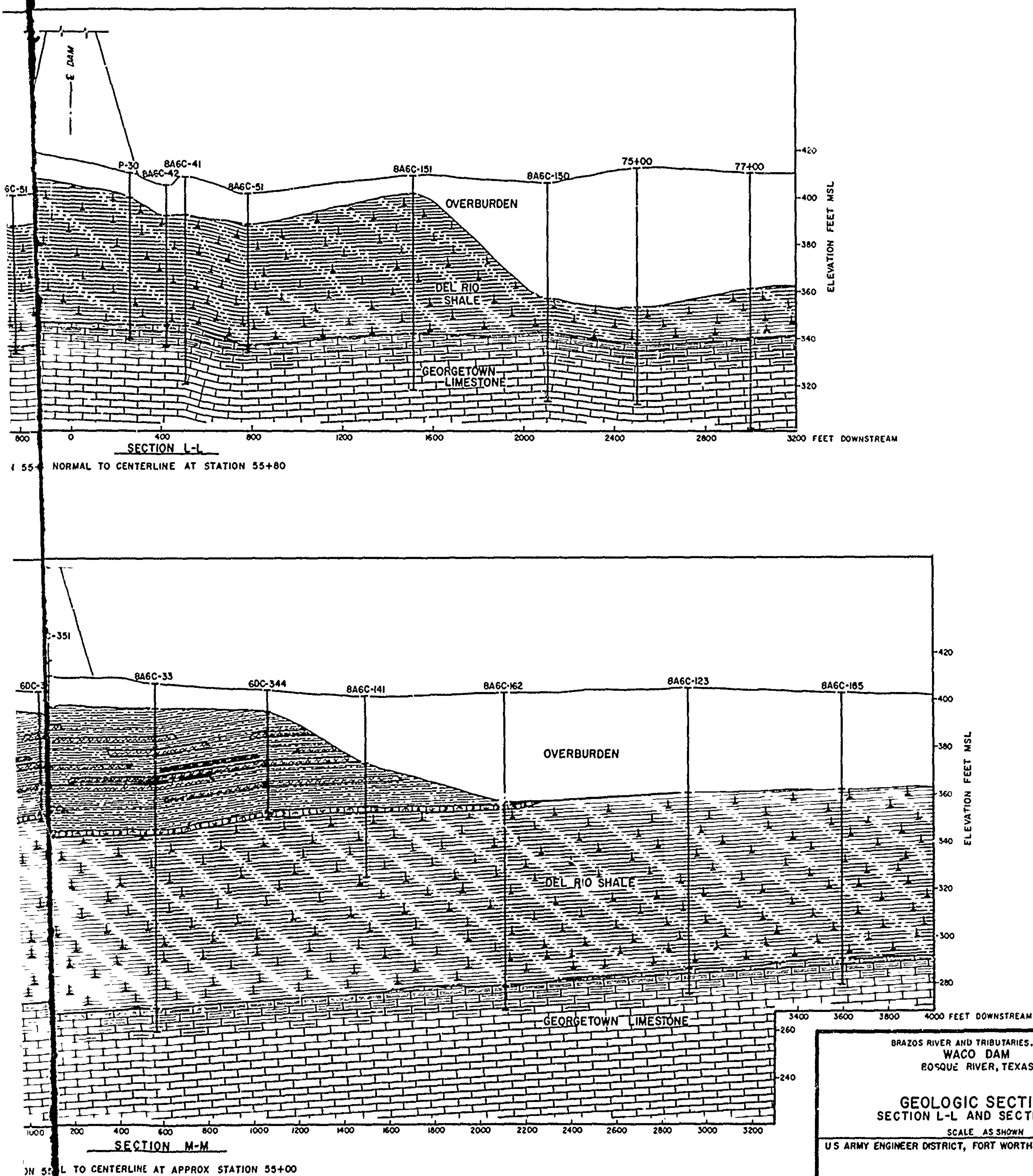
SECTION L-L

NORMAL TO CENTERLINE AT STATION 55



SECTION M-M

NORMAL TO CENTERLINE AT APPROX. STATION 5



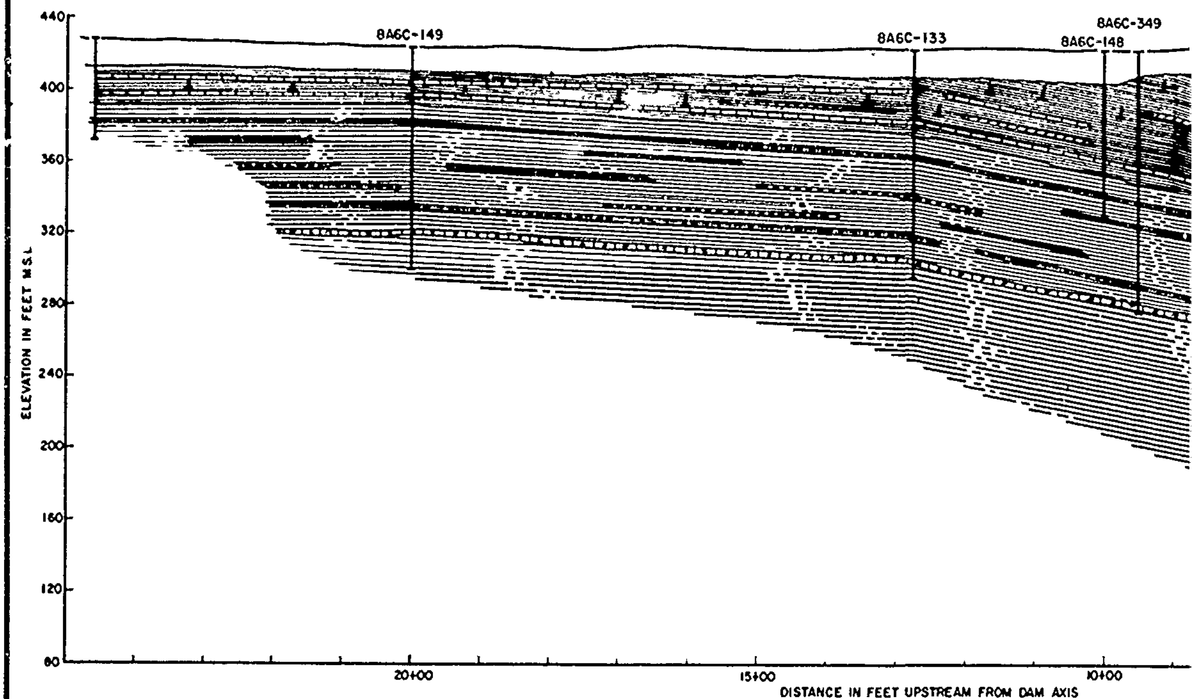
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

GEOLOGIC SECTIONS
SECTION L-L AND SECTION M-M

SCALE AS SHOWN

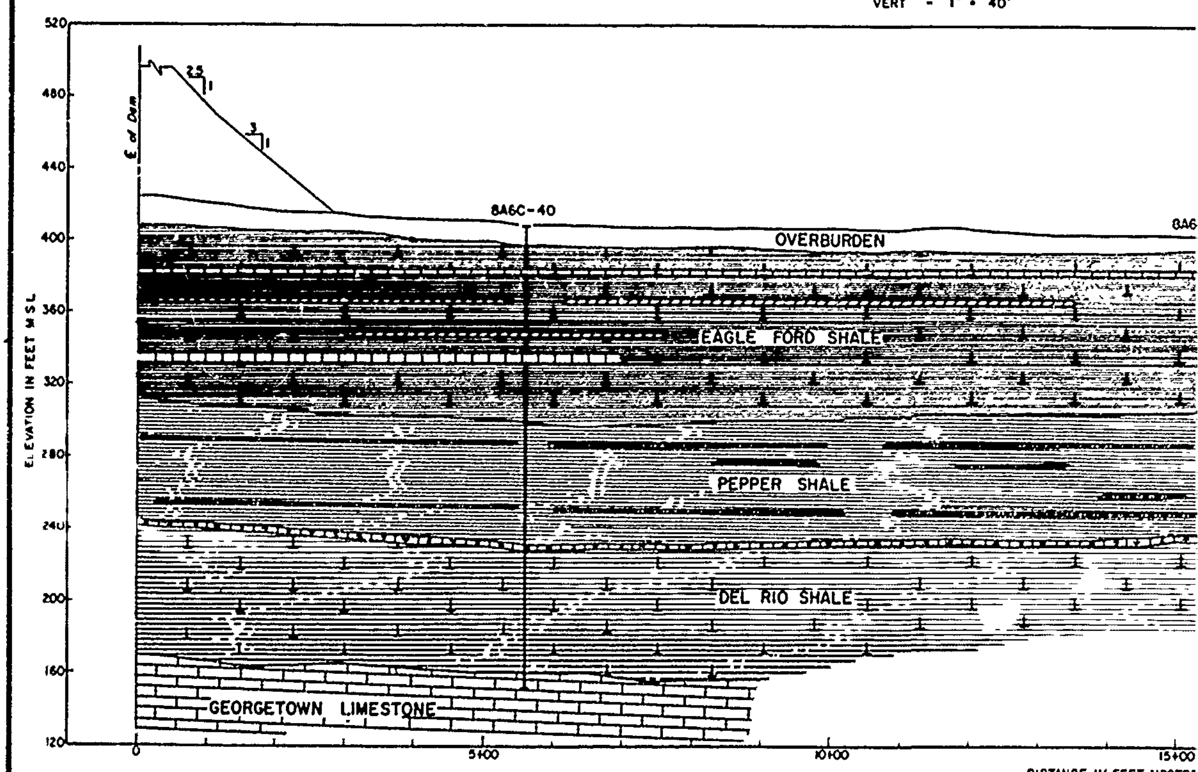
US ARMY ENGINEER DISTRICT, FORT WORTH JAN 1963

CORPS OF ENGINEERS



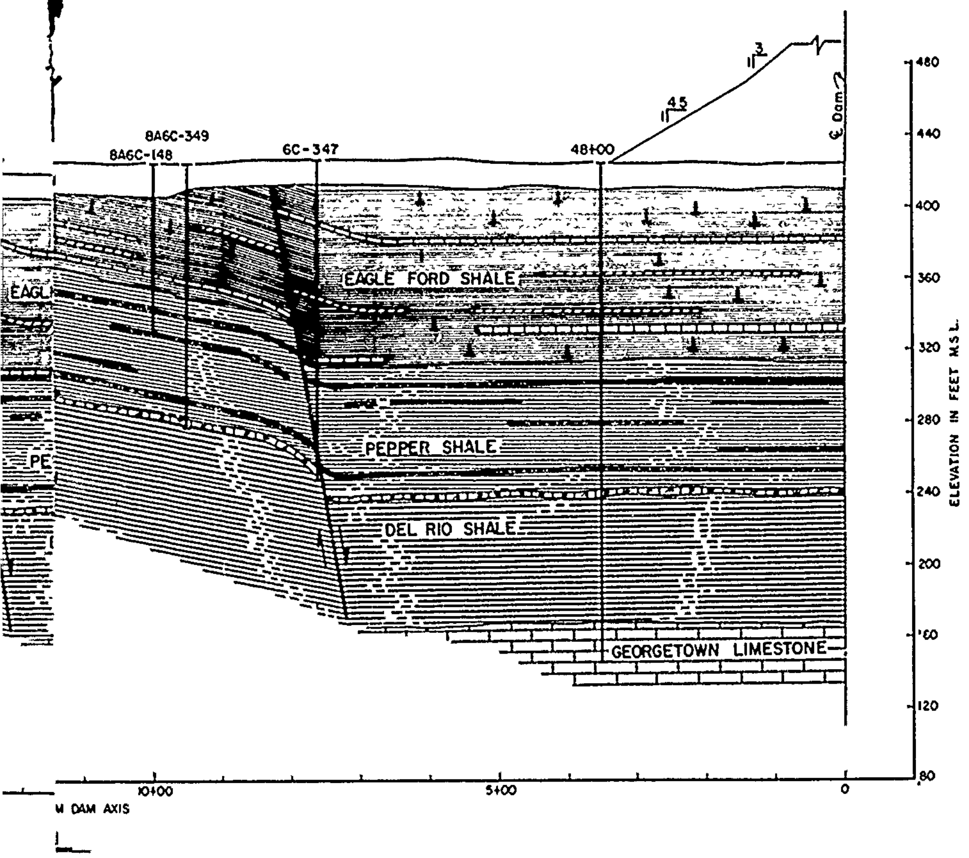
SECTION N-N

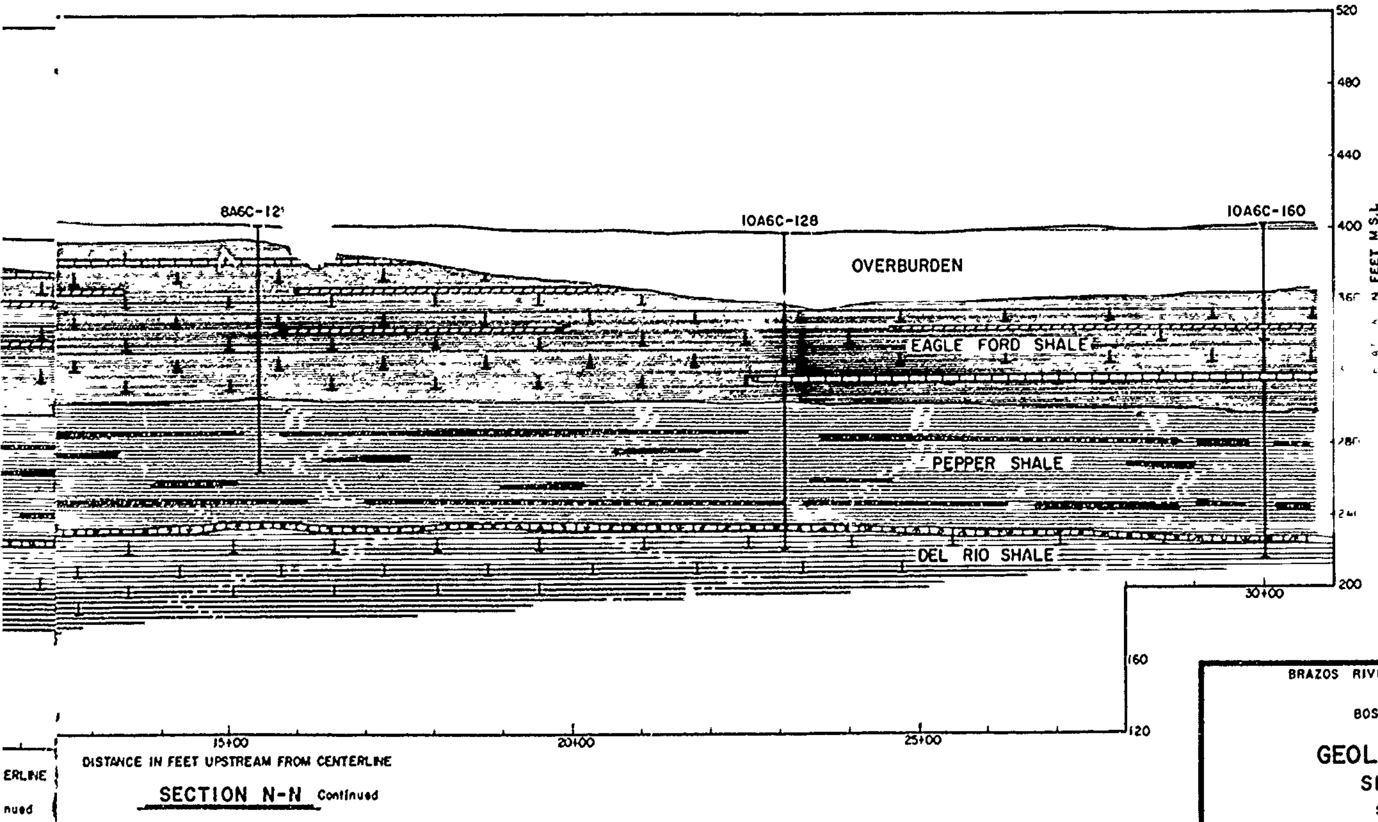
SCALE
HORIZ. - 1" = 100'
VERT - 1" = 40'



SECTION

U.S. ARMY



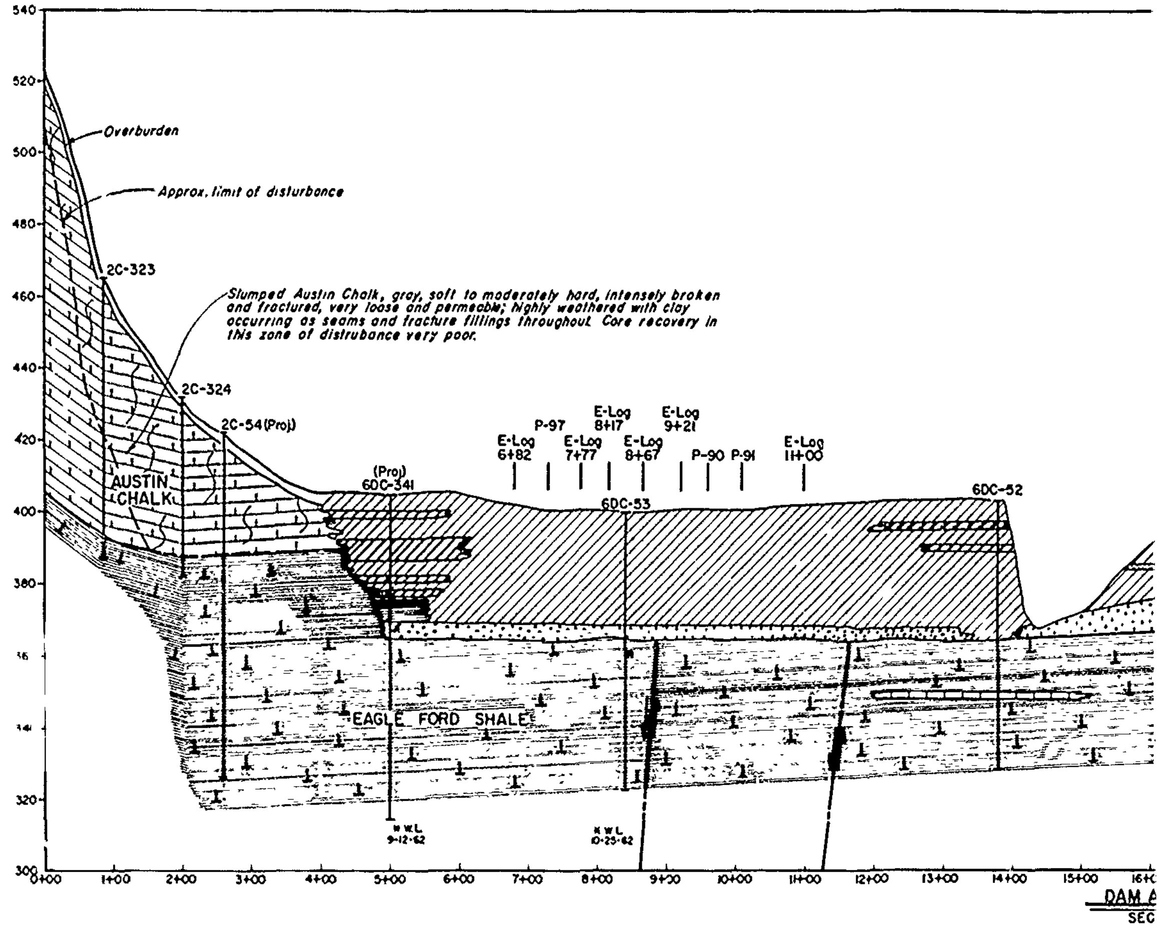


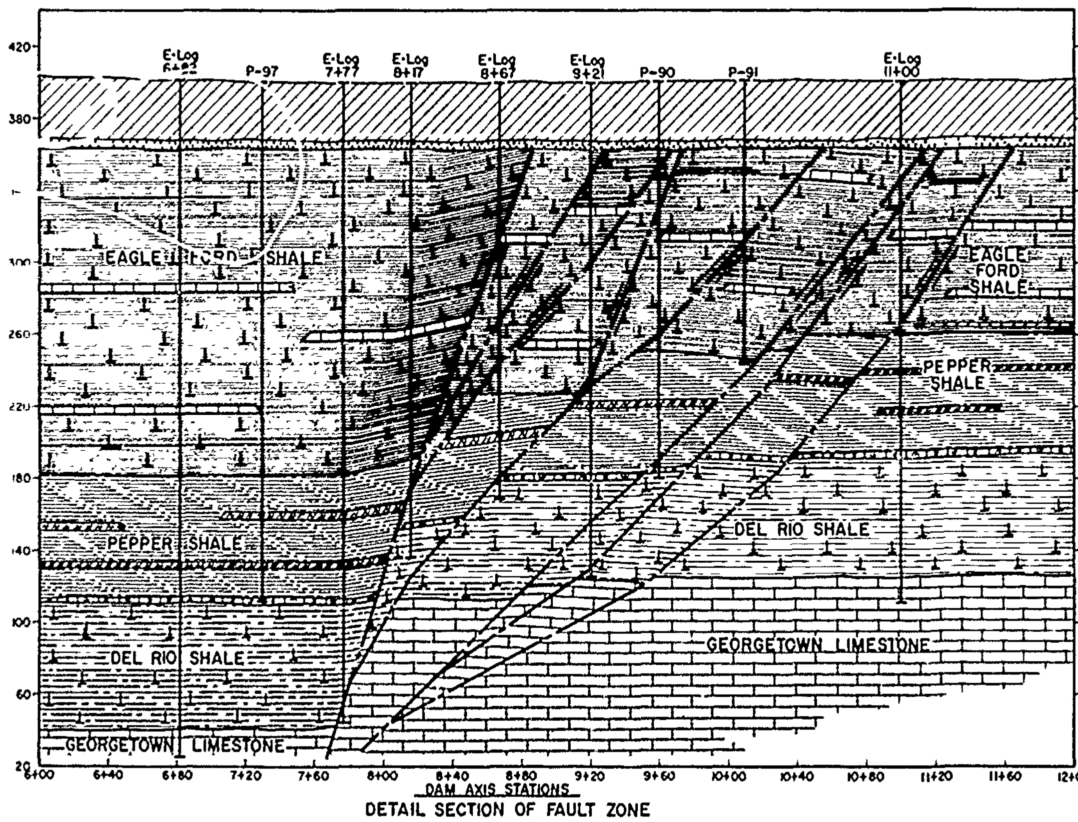
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

GEOLOGIC SECTION
SECTION N-N
SCALE AS SHOWN

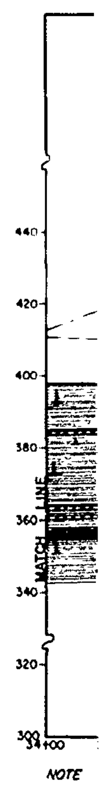
U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN 1963

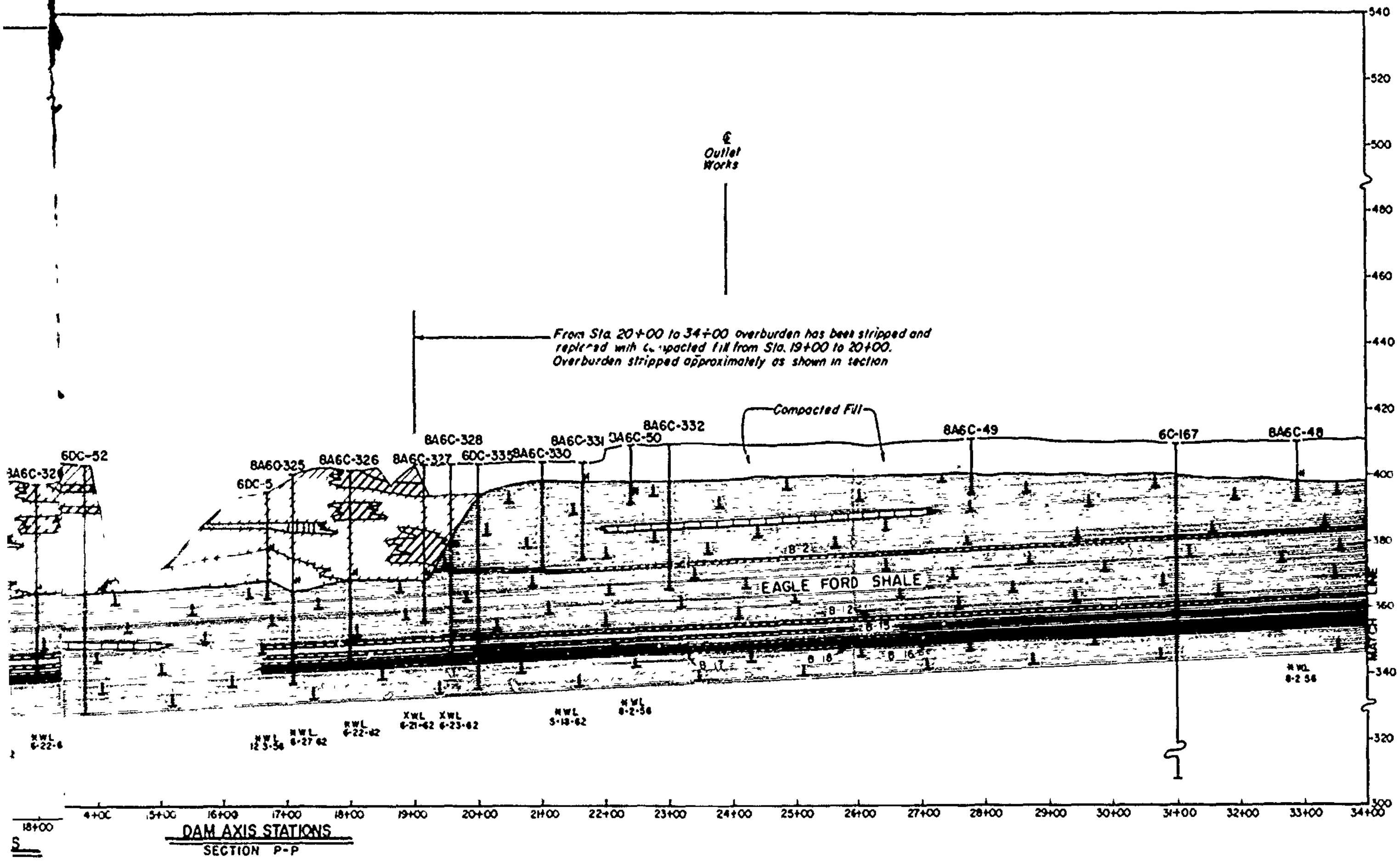
PLATE 33

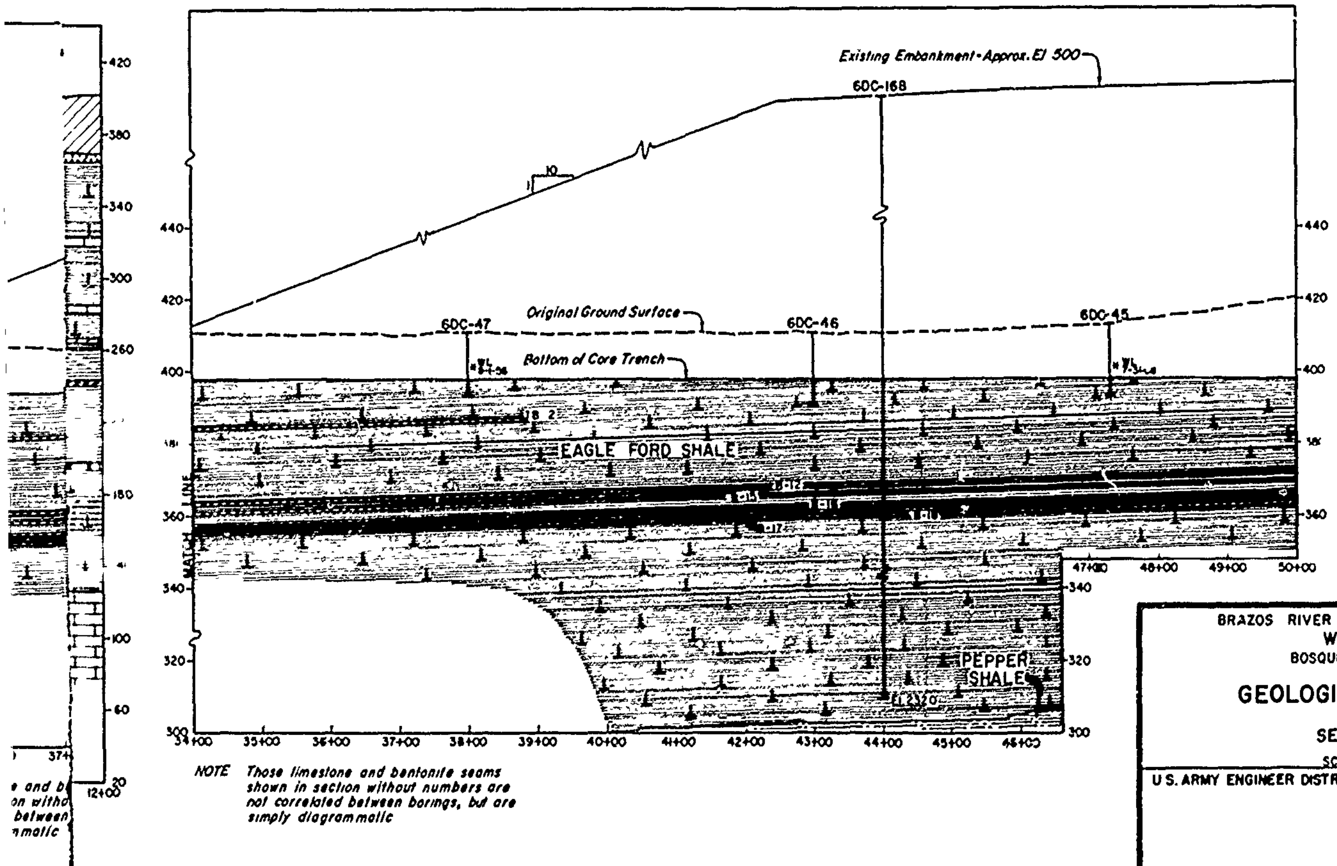




DETAIL SECTION OF FAULT ZONE







NOTE Those limestone and bentonite seams shown in section without numbers are not correlated between borings, but are simply diagrammatic

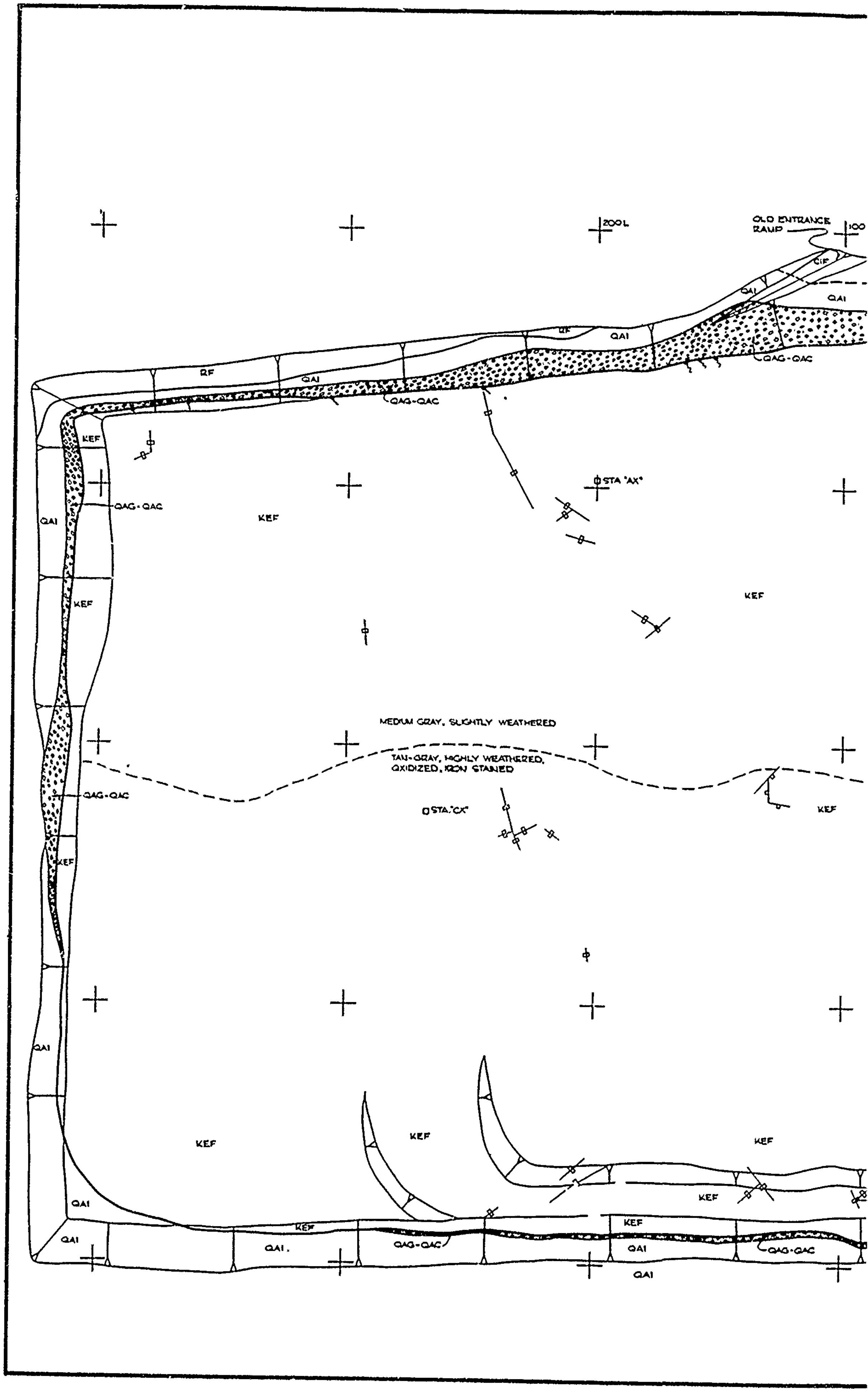
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

GEOLOGIC SECTIONS

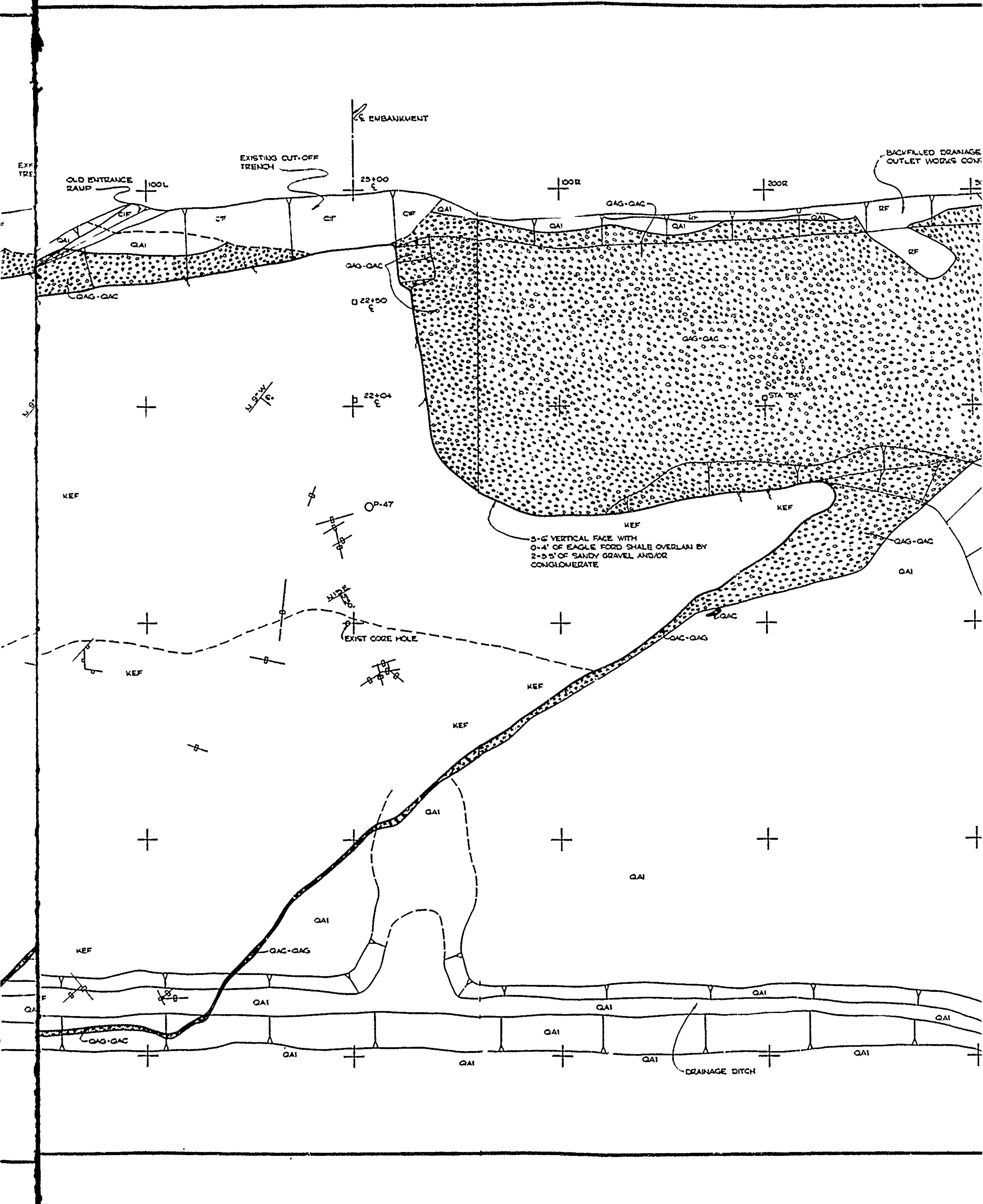
SECTION P-P

SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH JAN. 1963



200L
OLD ENTRANCE RAMP
100
CIF
QAI
RF
QAG-QAC
KEF
STA "AX"
MEDIUM GRAY, SLIGHTLY WEATHERED
TAN-GRAY, HIGHLY WEATHERED, OXIDIZED, IRON STAINED
STA "CX"



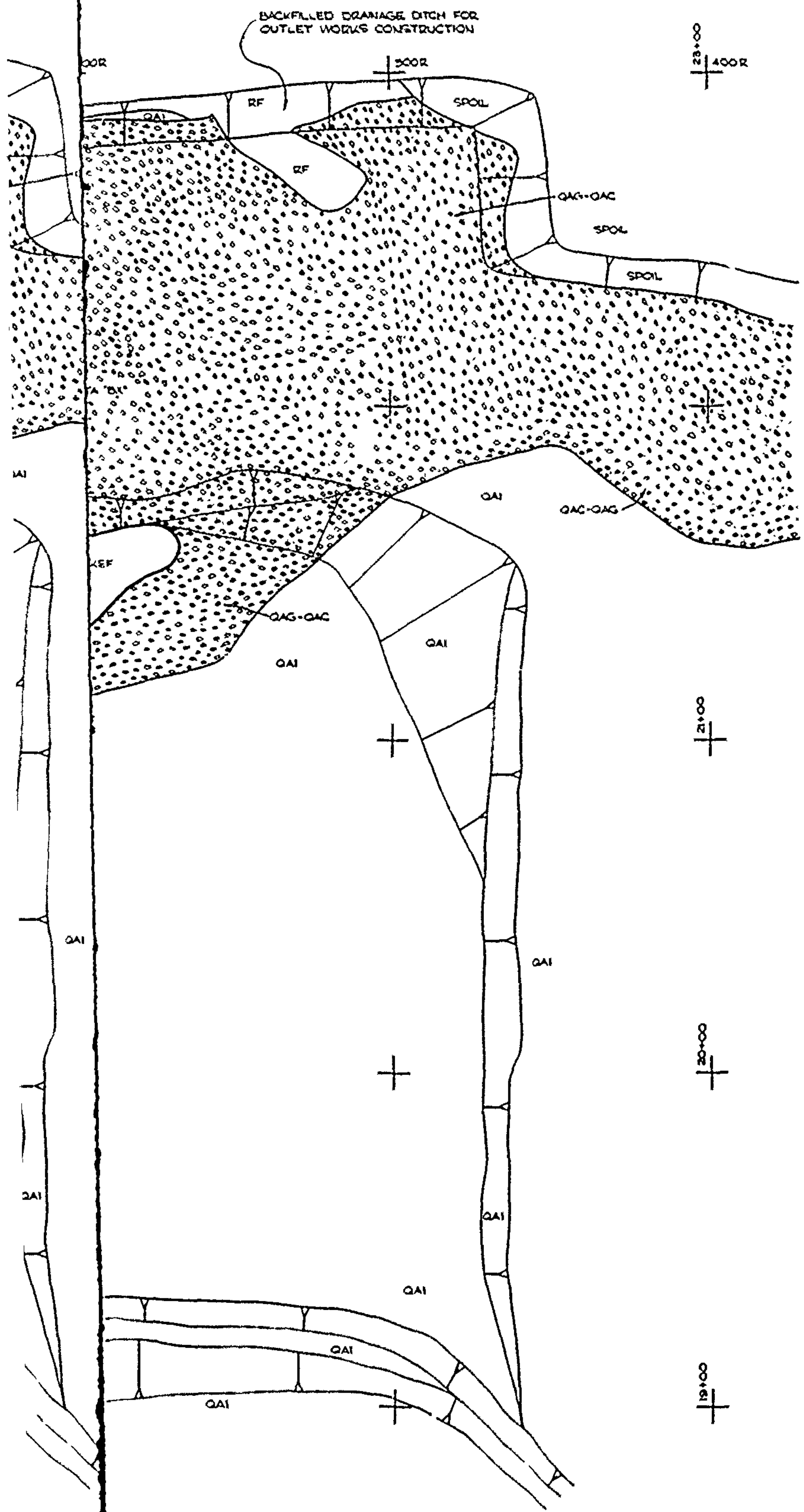
℄ EMBANKMENT
EXISTING CUT-OFF TRENCH
OLD ENTRANCE RAMP
100L
25+00 ℄
100R
200R
BACKFILLED DRAINAGE
CIF
QAI
QAG-QAC
RF
22+50 ℄
22+04 ℄
STA "BX"
KEF
P-47
5-6' VERTICAL FACE WITH
0-4' OF EAGLE FORD SHALE OVERLAIN BY
2-3.5' OF SANDY GRAVEL AND/OR
CONGLOMERATE
EXIST CORE HOLE
QAC
QAC-QAG
DRAINAGE DITCH

SPOIL WASTE MATERIALS FROM CONSTRUCTION OF OUTLET WORKS
CIF COMPACTED IMPERVIOUS FILL
RF RANDOM FILL
QAG QUATERNARY ALLUVIAL GRAVELS
QAC QUATERNARY ALLUVIAL CONGLOMERATE
QAI QUATERNARY ALLUVIUM (SANDS, CLAYS ETC.)
KEF CRETACEOUS, EAGLE FORD SHALE

VERTICAL JOINT

WATER SEEP

STA. "BX" PLANE TABLE STATION



BRA
G
U.S. ARMY ENG

STE MATERIALS FROM CONSTRUCTION OF OUTLET WORKS
MPACTED IMPERVIOUS FILL
NDOM FILL
UATERNARY ALLUVIAL GRAVELS
UATERNARY ALLUVIAL CONGLOMERATE
UATERNARY ALLUVIUM (SANDS, CLAYS, ETC.)
ETACEOUS, EAGLE FORD SHALE

RTICAL JOINT

TER SEEP

ANE TABLE STATION

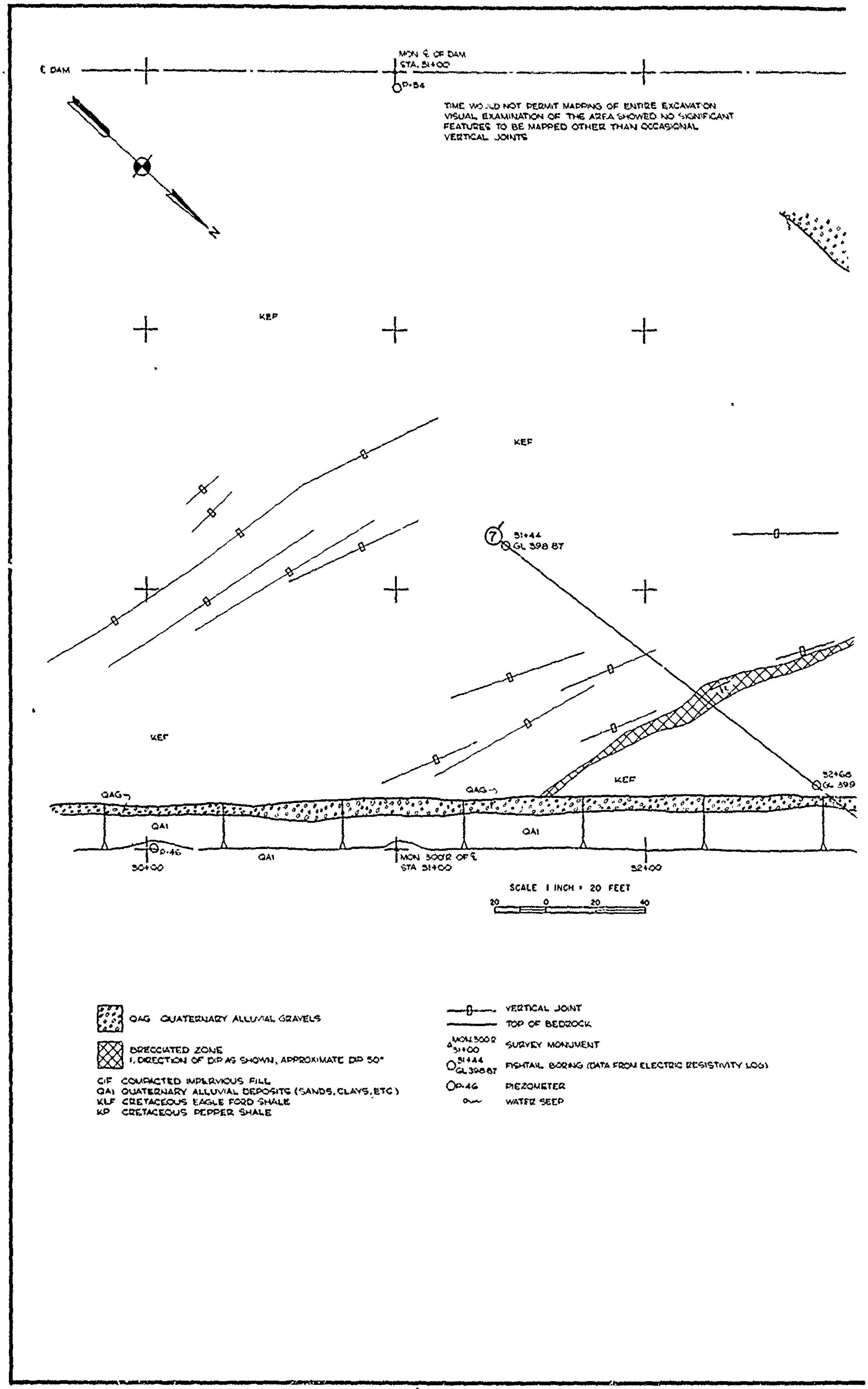
ND TRIBUT
O DAM
RIVER, TE
F EMBA
NDATIO
OO TO 2
AS SHOWN
, FORT WO

BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

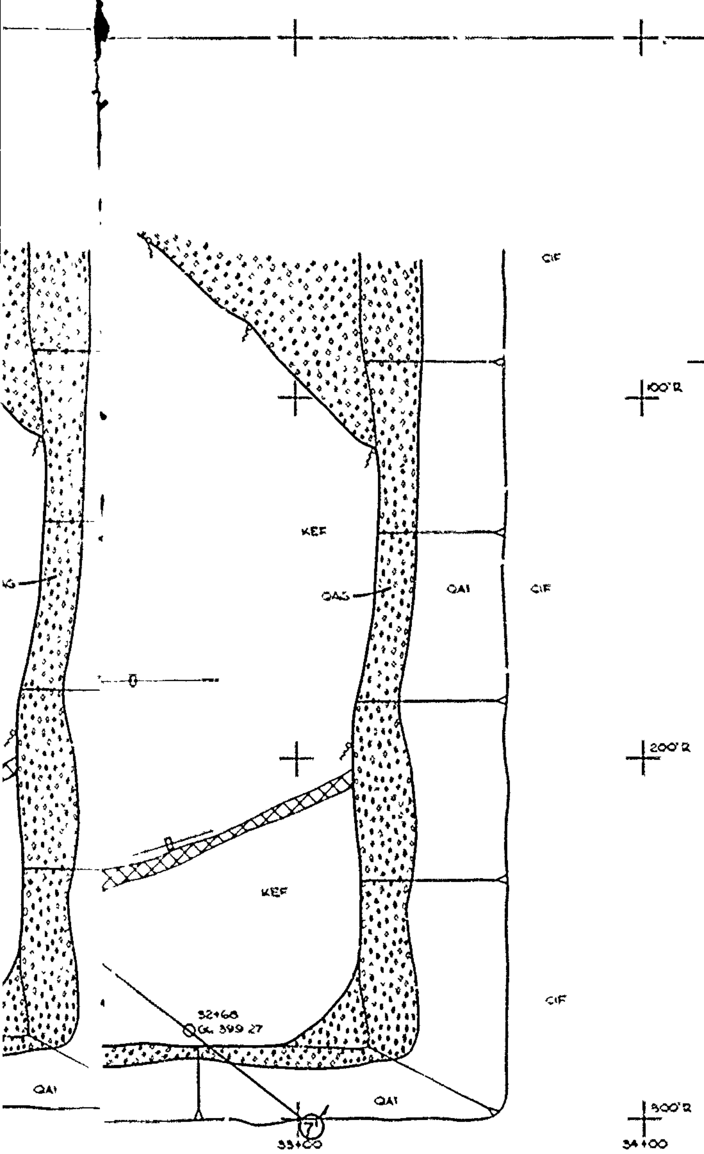
GEOLOGY OF EMBANKMENT FOUNDATION

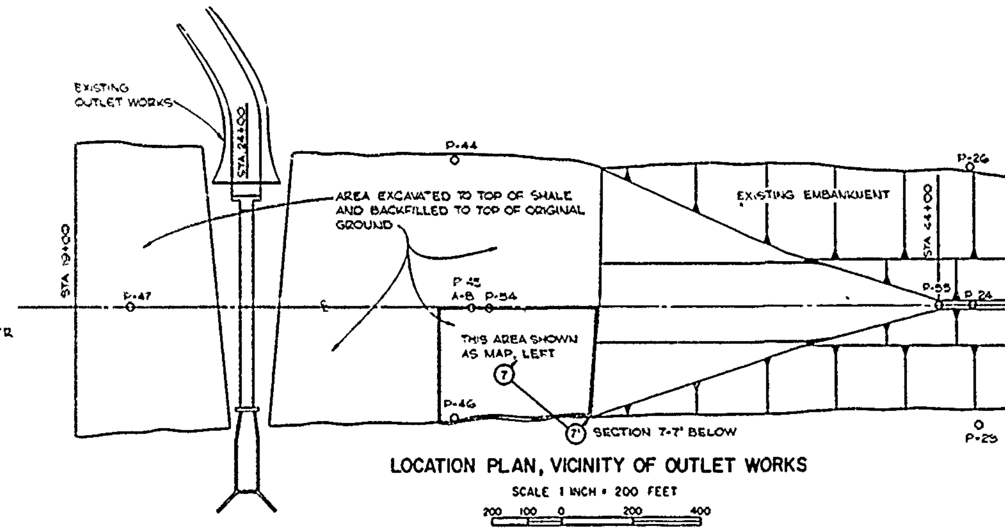
STA. 19+00 TO 23+00
SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN 1963



℄ DAM
MON ℄ OF DAM
STA. 51+00
P-54
TIME WOULD NOT PERMIT MAPPING OF ENTIRE EXCAVATION
VISUAL EXAMINATION OF THE AREA SHOWED NO SIGNIFICANT
FEATURES TO BE MAPPED OTHER THAN OCCASIONAL
VERTICAL JOINTS
N
KEF
KEF
51+44
GL 398.87
KEF
KEF
52+68
GL 399
QAG
QAG
QAl
QAl
QAl
P-46
MON 300'R OF ℄
STA 51+00
50+00
52+00
SCALE 1 INCH = 20 FEET
20
0
20
40
QAG QUATERNARY ALLUVIAL GRAVELS
BRECCIATED ZONE
1, DIRECTION OF DIP AS SHOWN, APPROXIMATE DIP 50°
CIF COMPACTED IMPERVIOUS FILL
QAl QUATERNARY ALLUVIAL DEPOSITS (SANDS, CLAYS, ETC)
KEF CRETACEOUS EAGLE FORD SHALE
KP CRETACEOUS PEPPER SHALE
VERTICAL JOINT
TOP OF BEDROCK
MON 300'R 51+00
SURVEY MONUMENT
51+44 GL 398.87
FISHTAIL BORING (DATA FROM ELECTRIC RESISTIVITY LOG)
P-46
PIEZOMETER
WATER SEEP

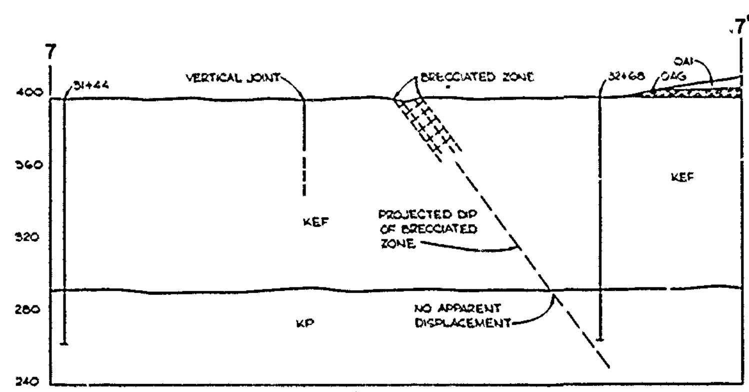




LOCATION PLAN, VICINITY OF OUTLET WORKS

SCALE 1 INCH = 200 FEET

200 100 0 200 400



SECTION 7-7'

SCALE HORIZ 1" = 20'
VERT 1" = 40'

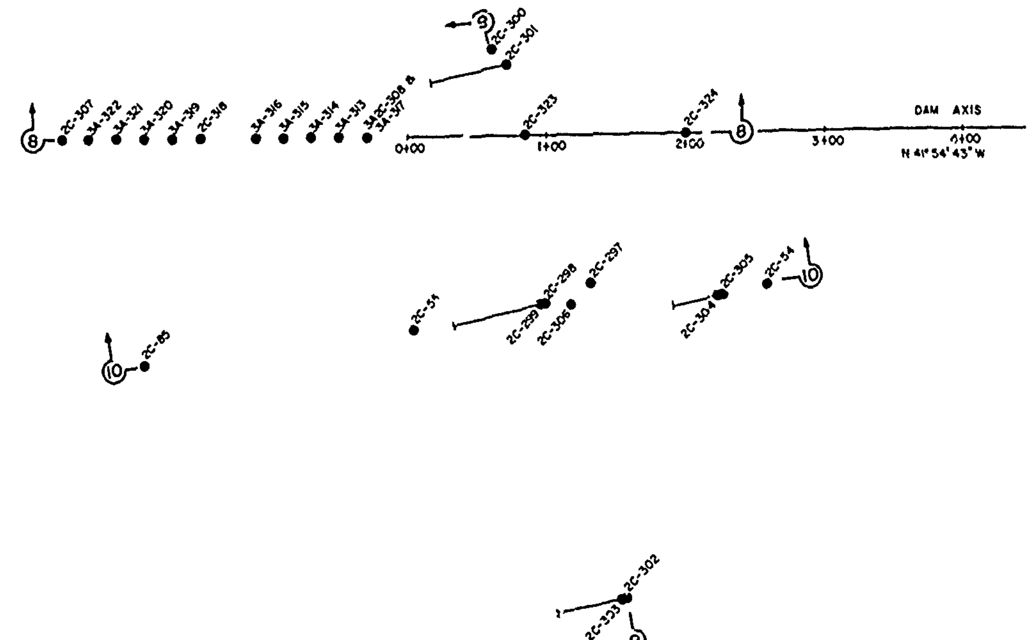
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

GEOLOGY OF EMBANKMENT FOUNDATION

STA. 25+00 TO 34+00

SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JUNE 1969



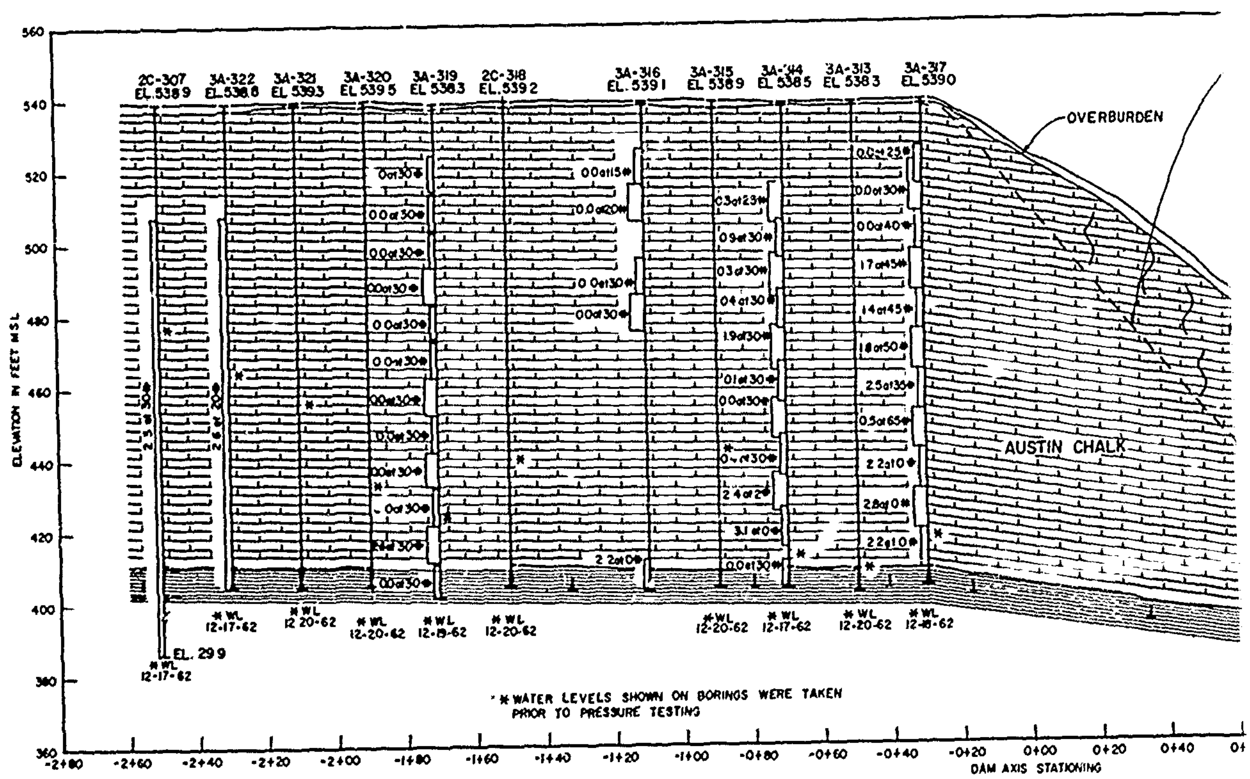
PLAN

DETAILED BORING LAYOUT

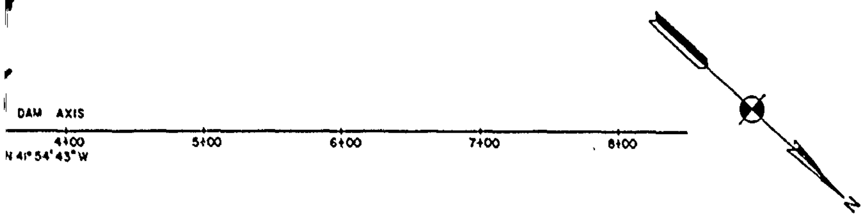
RIGHT ABUTMENT

SCALE OF FEET

50 0 50 100



SECTION 8-8



LEGEND

8A 6C-325 — EXPLORATION BORING prefixes before number indicates type and size of boring.

P-20 — PIEZOMETER

E-LOG 11+00 — EXPLORATION BORING-Drilled at dam axis station 11+00 for electric logging

SLUMPED AUSTIN CHALK: Gray, soft, intensely broken and fractured, highly weathered.

AUSTIN CHALK Tan-gray to light-gray, moderately hard marly chalk.

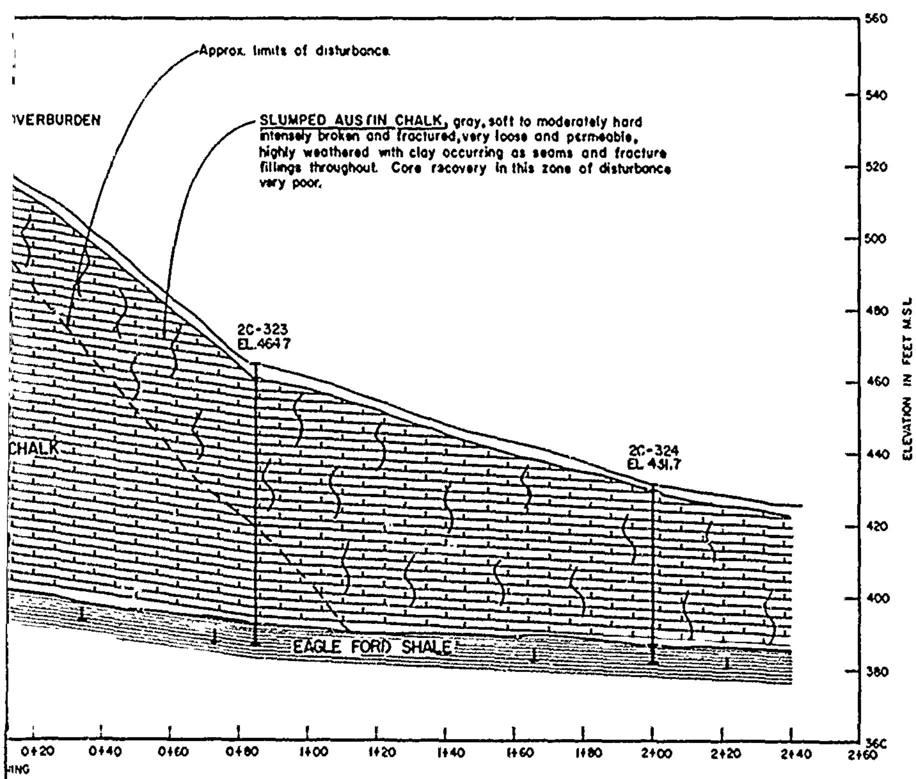
EAGLE FORD SHALE: Dark gray to black, soft to moderately hard shale with bentonite and limestone seams in the lower section. Numbered bentonite seams are correlatable between borings

PEPPER SHALE: Black, soft, compaction shale with siltstone and sandy siltstone seams

DEL RIO SHALE. Greenish gray, soft, calcareous compaction shale.

GEORGETOWN LIMESTONE: Light gray, hard, argillaceous limestone.

NOTE. Overburden symbols are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM
Asterisk (*) to right of borings indicates level of ground water in those borings where obtained. Date of reading is shown at bottom of boring as *W L 8-2-56.
The absence of ground water levels opposite certain borings does not necessarily mean that ground water will not be encountered at the locations or within the vertical reaches of those borings.

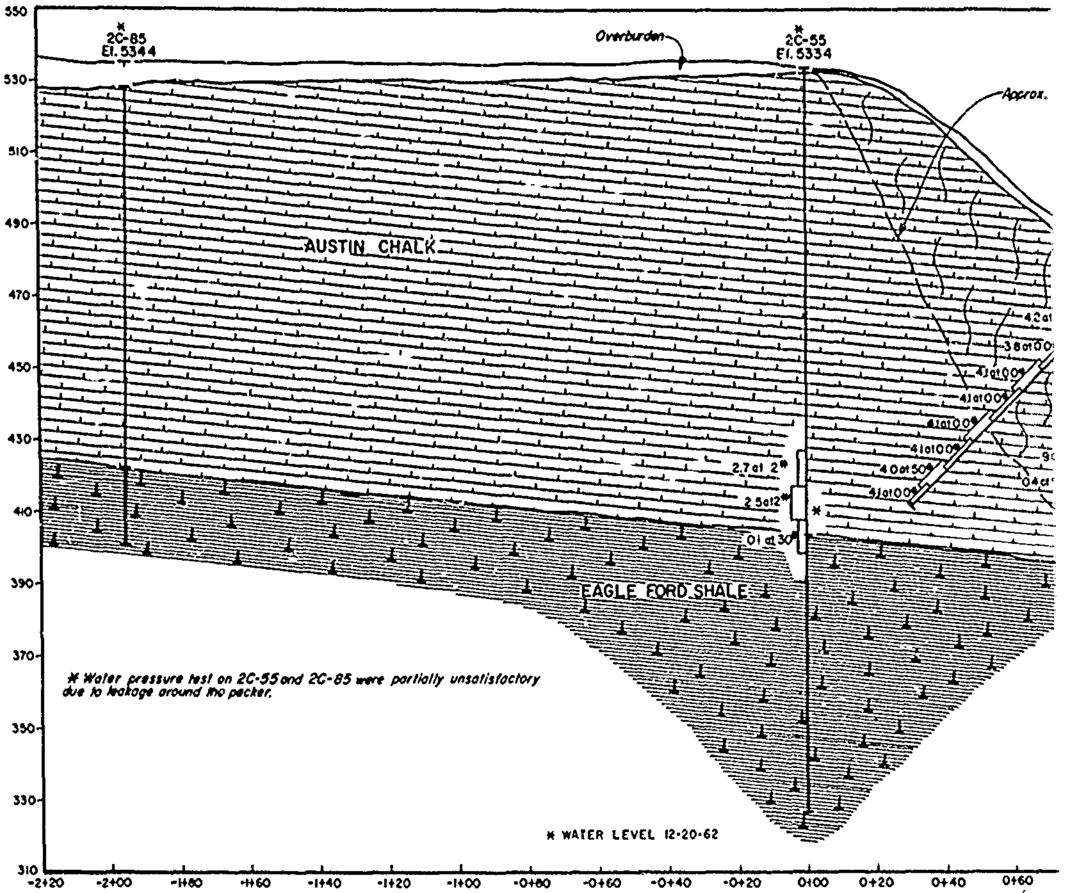


NOTE:
SECTION LOCATED ALONG DAM AXIS LOOKING UPSTREAM

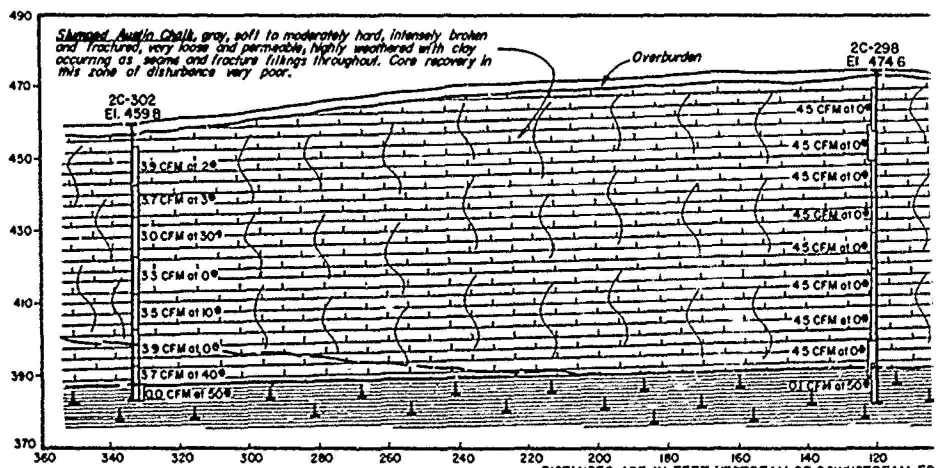
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

GEOLOGIC SECTION AND
PLAN OF BORINGS
RIGHT ABUTMENT

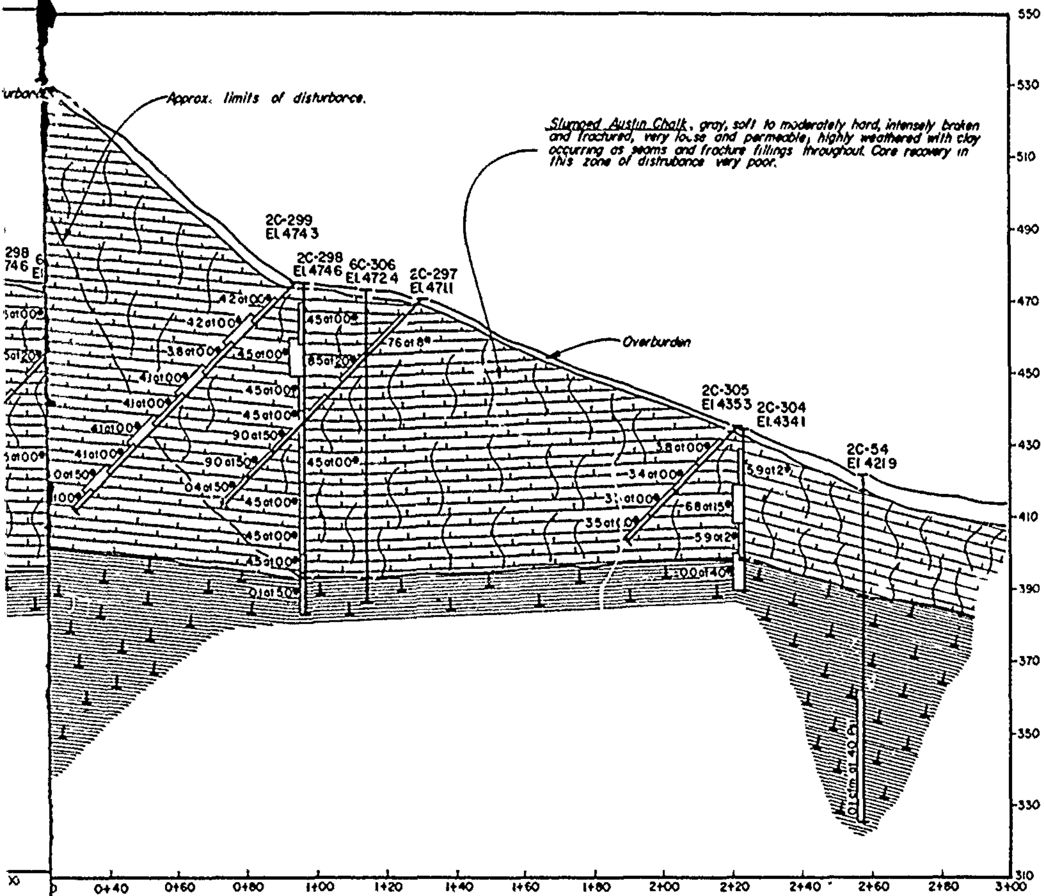
U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JUNE 1959



DAM AXIS STATIO
SECTION LOCATED PARALLEL TO DAM AXIS
SECTION 10-



DISTANCES ARE IN FEET UPSTREAM OR DOWNSTREAM FR
SECTION LOCATED PERPENDICULAR TO DAM AXIS AT ST
SECTION 9-9



DAM AXIS STATIONING
PARALLEL TO DAM AXIS APPROX 120' DOWNSTREAM
SECTION 10-10

LEGEND

See Plate 34 for Description of Materials

2C-54 — 2" Core Boring

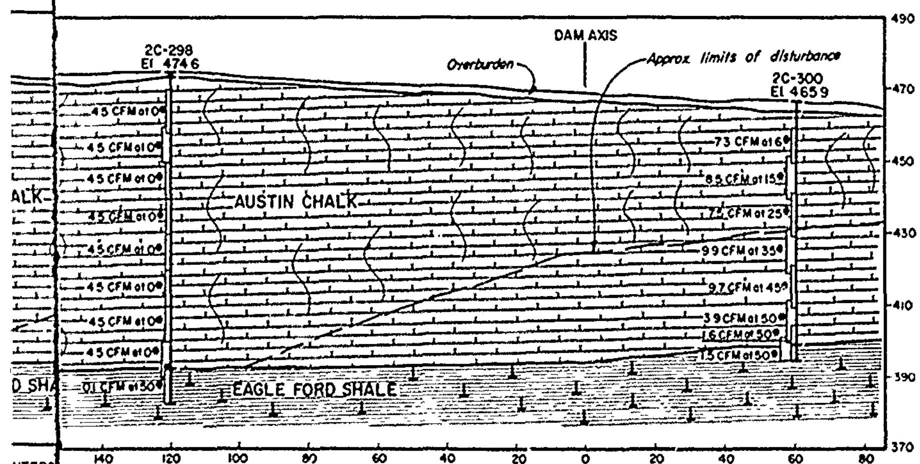
6C-306 — 6" Core Boring

Angle Boring

PRESSURE TEST RESULTS

45 at 2* — 45 Cubic Feet Per Minute Leakage at 2 Pounds Per Square Inch Guage Pressure Bracket Indicate Interval Tested

NOTE Where water pressure test show no guage pressure with takes up to 45 C F M, pump capacity was reached



AM OR DOWNSTREAM FROM DAM AXIS INTERSECTION.
LAR TO DAM AXIS AT STATION 0+75
DOWNSTREAM ——|—— UPSTREAM
SECTION 9-9

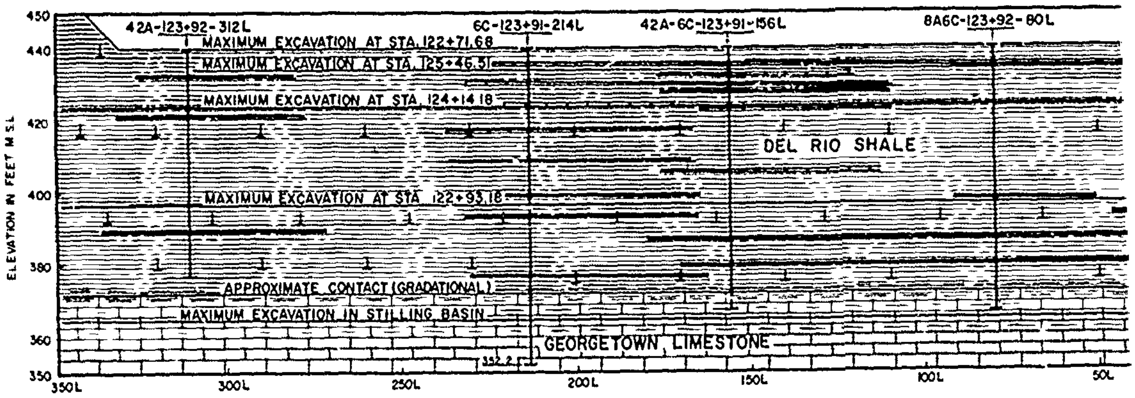
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

GEOLOGIC SECTIONS

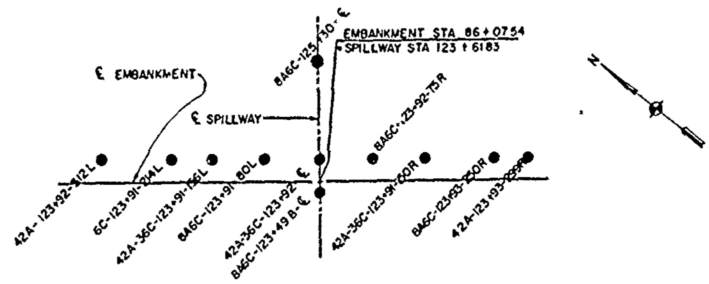
RIGHT ABUTMENT

SCALE AS SHOWN

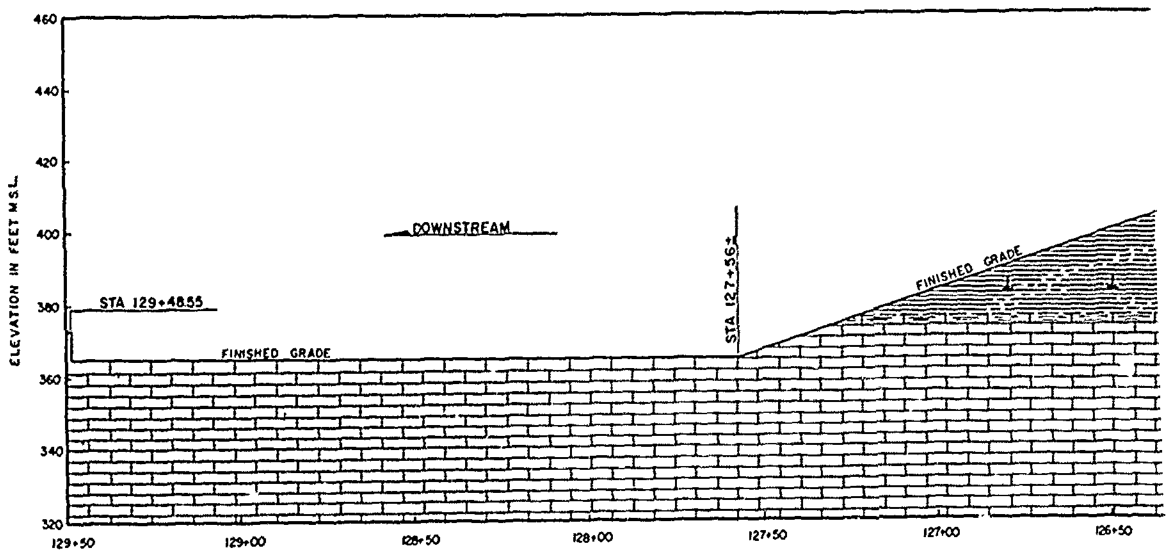
U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JUNE 1969



SE(



BORING LAYOUT



GEOLOGICAL PR(

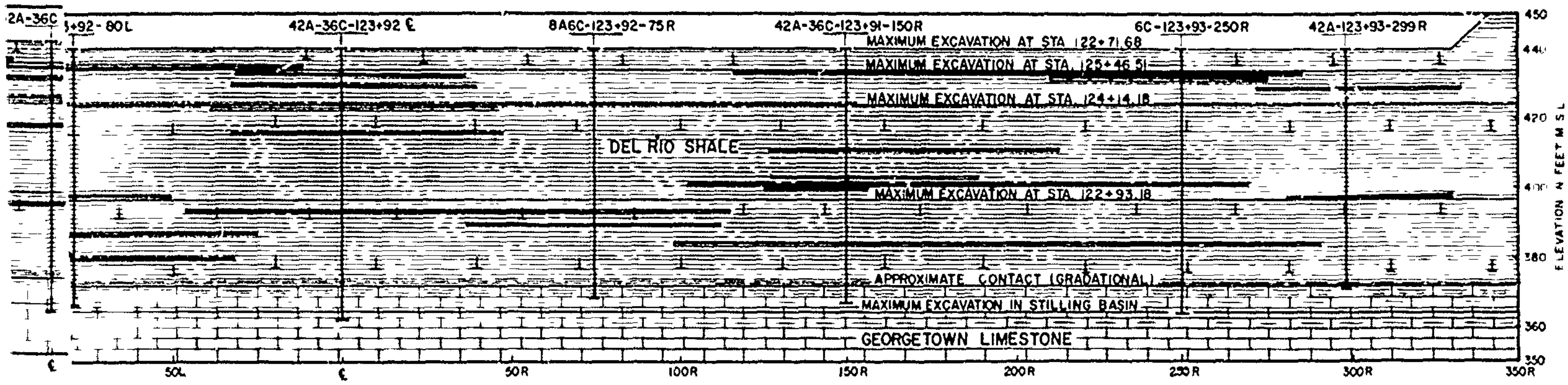
LEGEND

DEL RIO SHALE - soft to moderately hard, thin bedded, calcareous, (a cementation shale) slakes very slowly, fossiliferous, medium gray to gray green and contains occasional thin limestone seams, marly seams and softer carbonaceous seams.

8A6C — 8" AUGER AND 6" CORE BORING

GEORGETOWN LIMESTONE - lies conformably under the DEL RIO SHALE with a gradational contact, hard, thin to medium bedded, argillaceous, flaggy fine grained, dense, solid, non slaking, light gray, fossiliferous, occasional calcareous shale seams and partings.

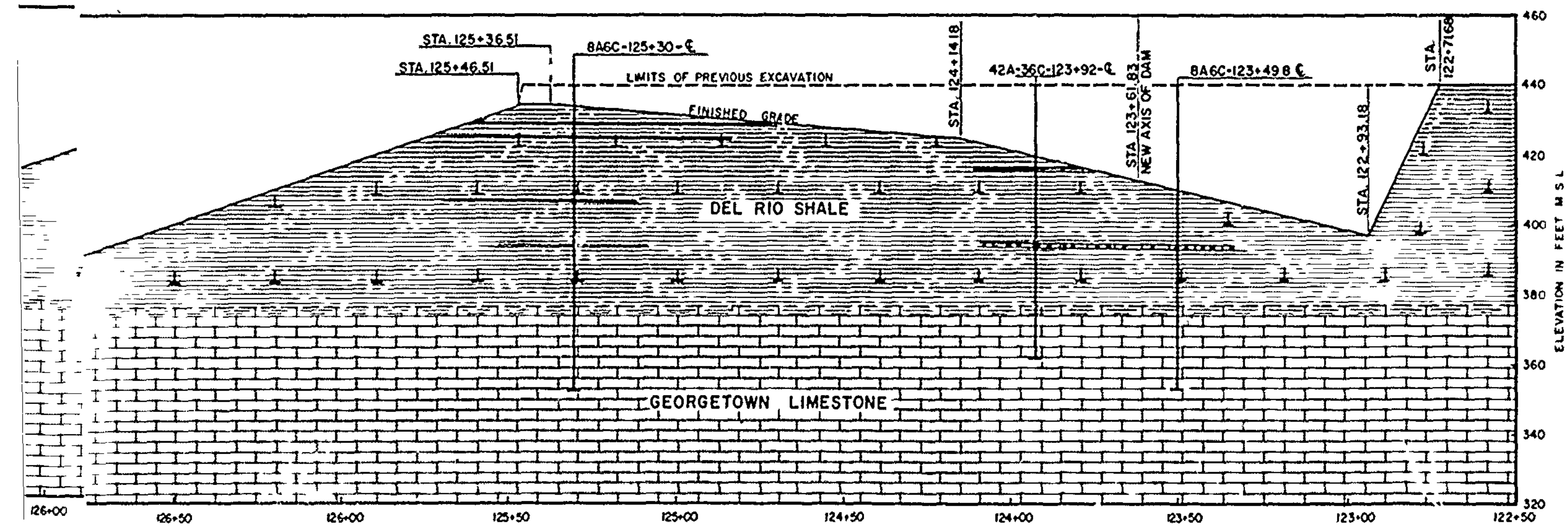
42A-36C — 42" AUGER AND 36" CALYX CORE BORING



SECTION AT SPILLWAY - STA. 123+90

(LOOKING DOWNSTREAM)

SCALE 1 INCH = 20 FEET



GEOLOGICAL PROFILE ALONG CENTERLINE OF SPILLWAY

SCALE 1 INCH = 20 FEET

BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

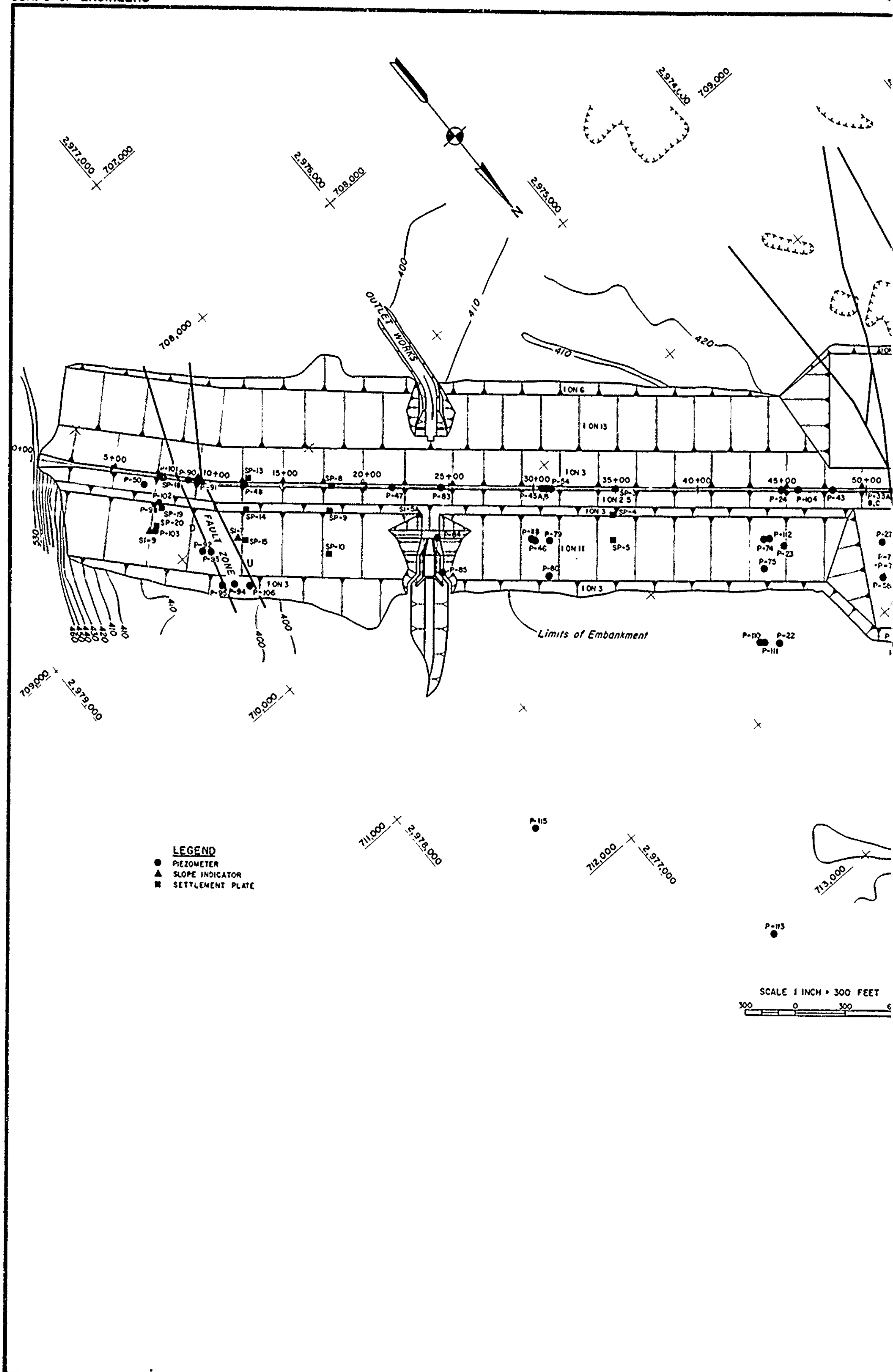
GEOLOGIC SECTIONS

SPILLWAY

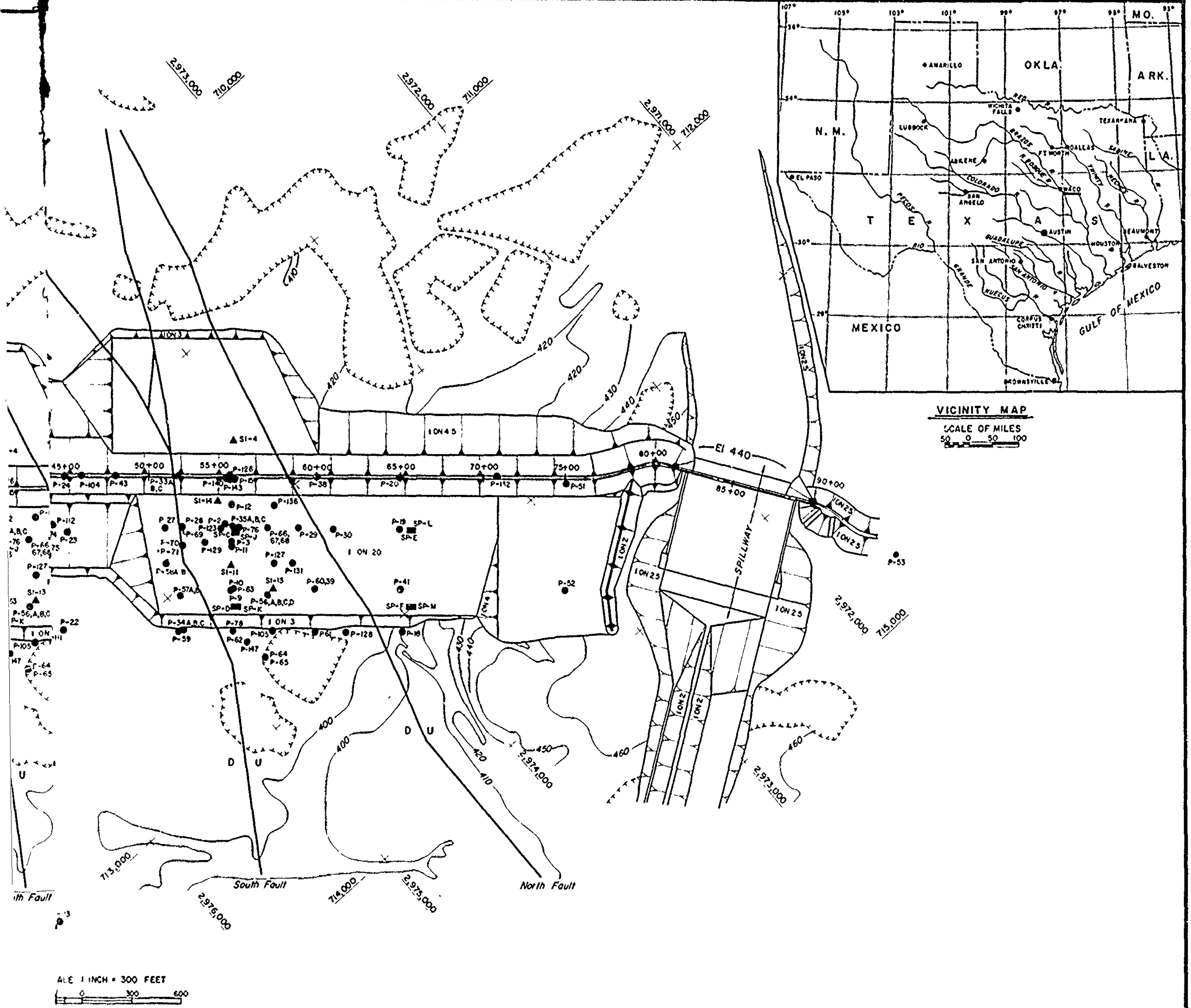
SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JUNE 1969

CORPS OF ENGINEERS



U.S. ARMY

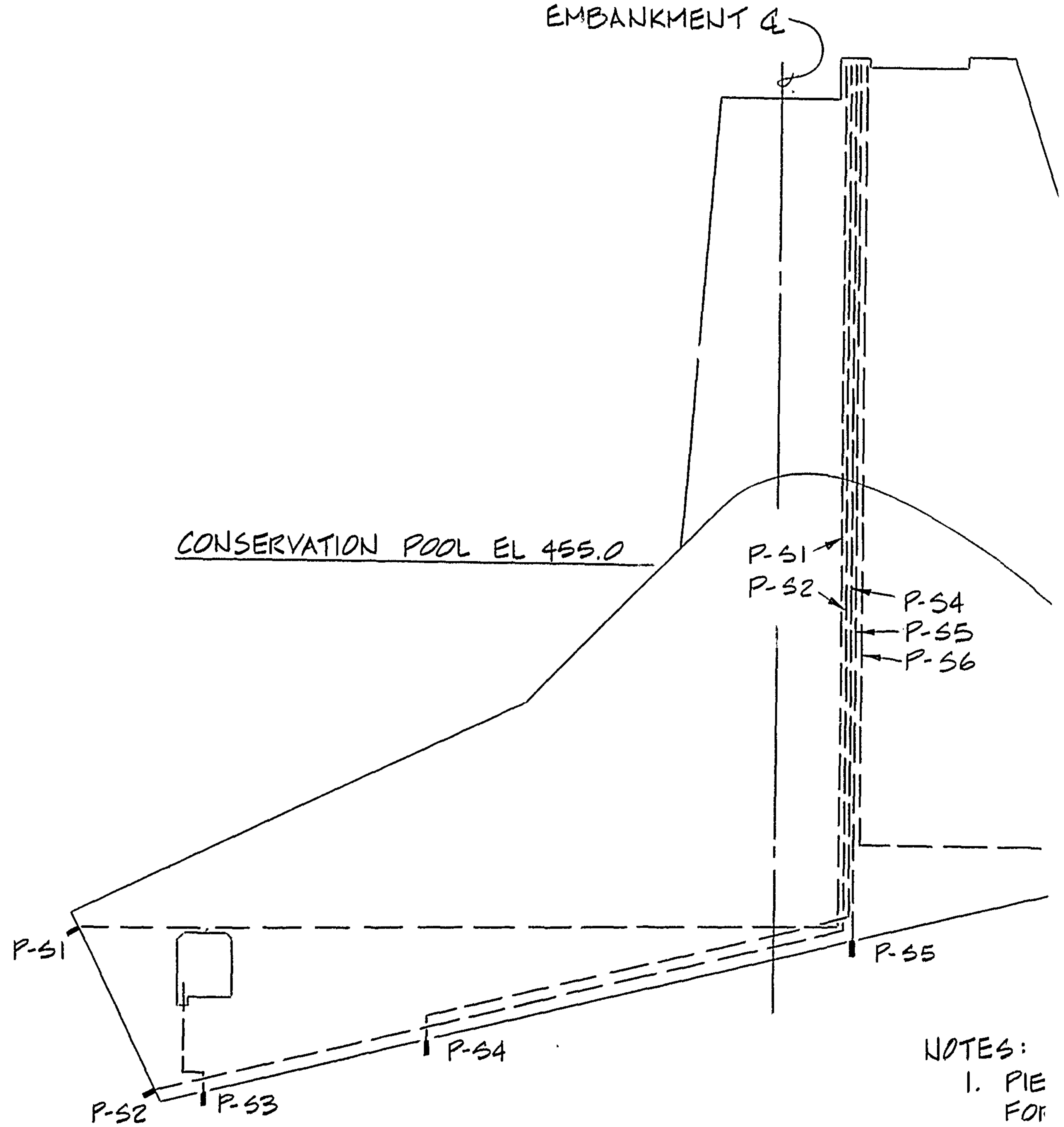


BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

FIELD INSTRUMENTATION PLAN

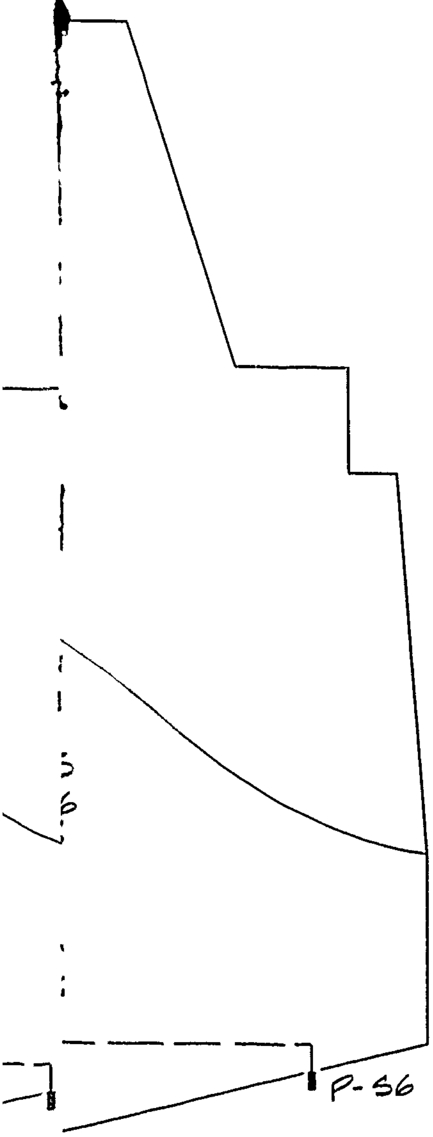
SCALES AS SHOWN

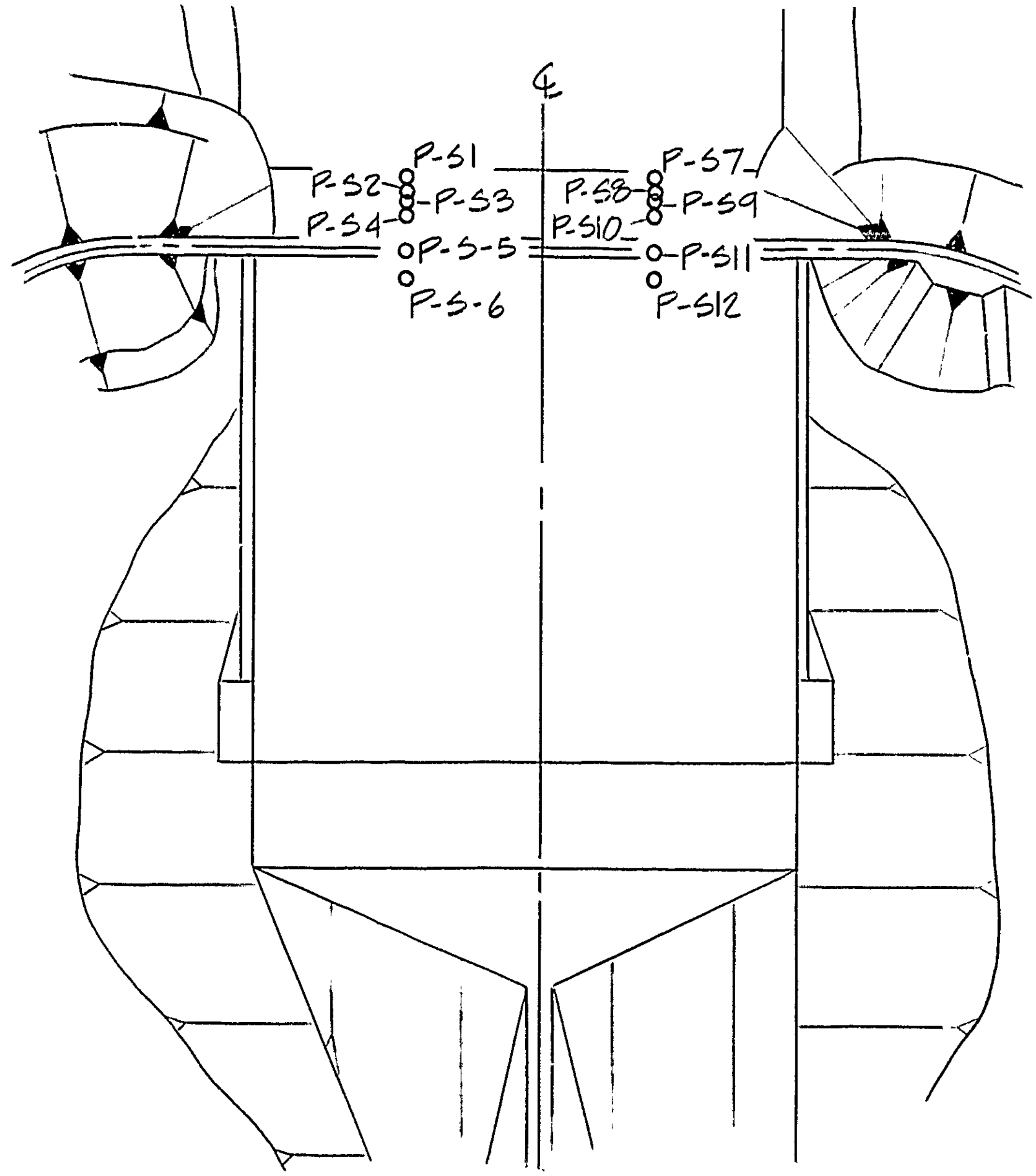
U.S. ARMY ENGINEER DISTRICT, FORT WORTH MAY 1968



NOTES:
1. PIE
FOR
2. PIE
ARE

SECTION
NTS





TES:

1. PIEZOMETER NUMBERS SHOWN ARE FOR PIER MONOLITH "G".
2. PIEZOMETERS IN PIER MONOLITH "N" ARE NUMBERED P-S7 THRU P-S12.

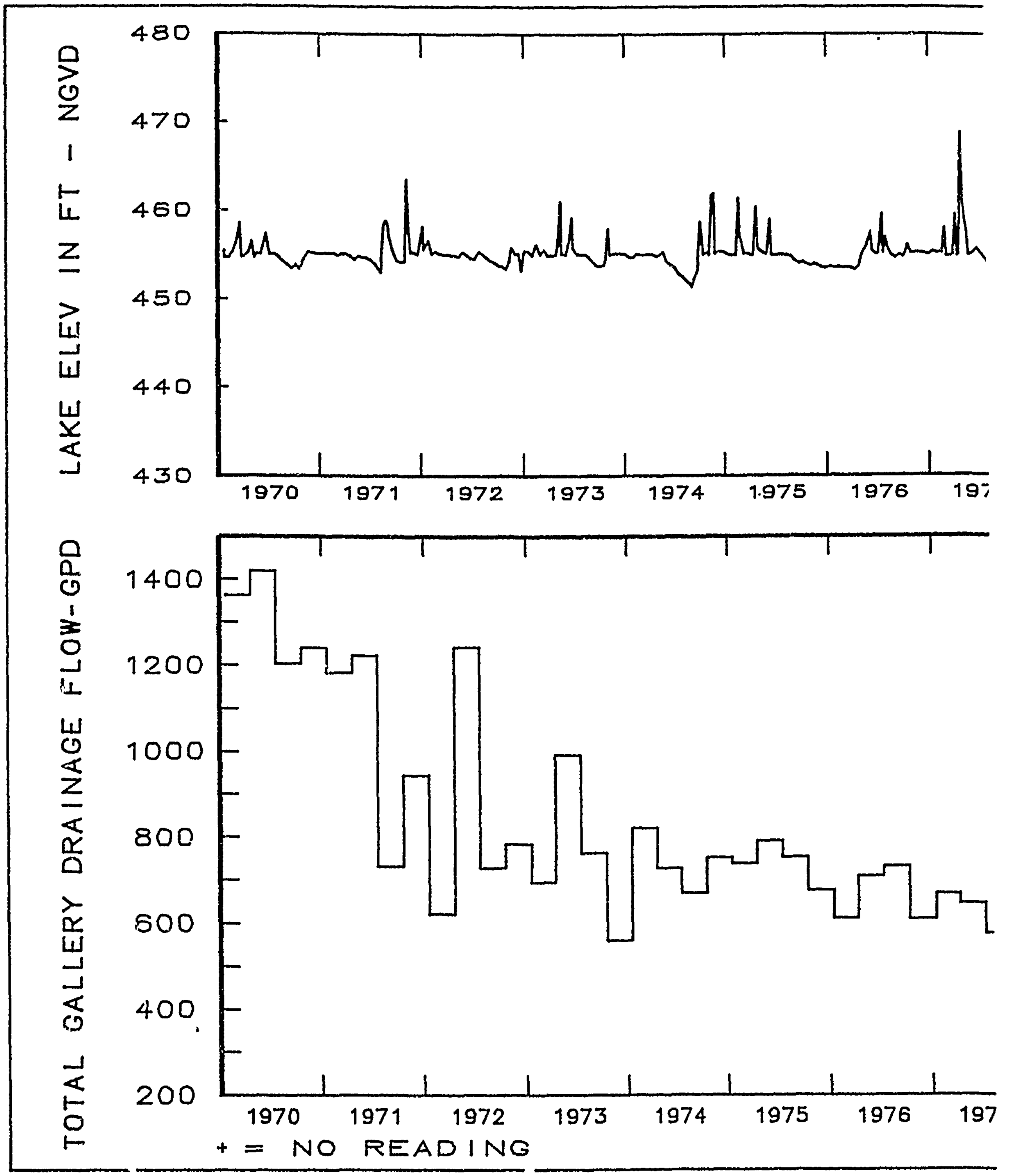
PLAN

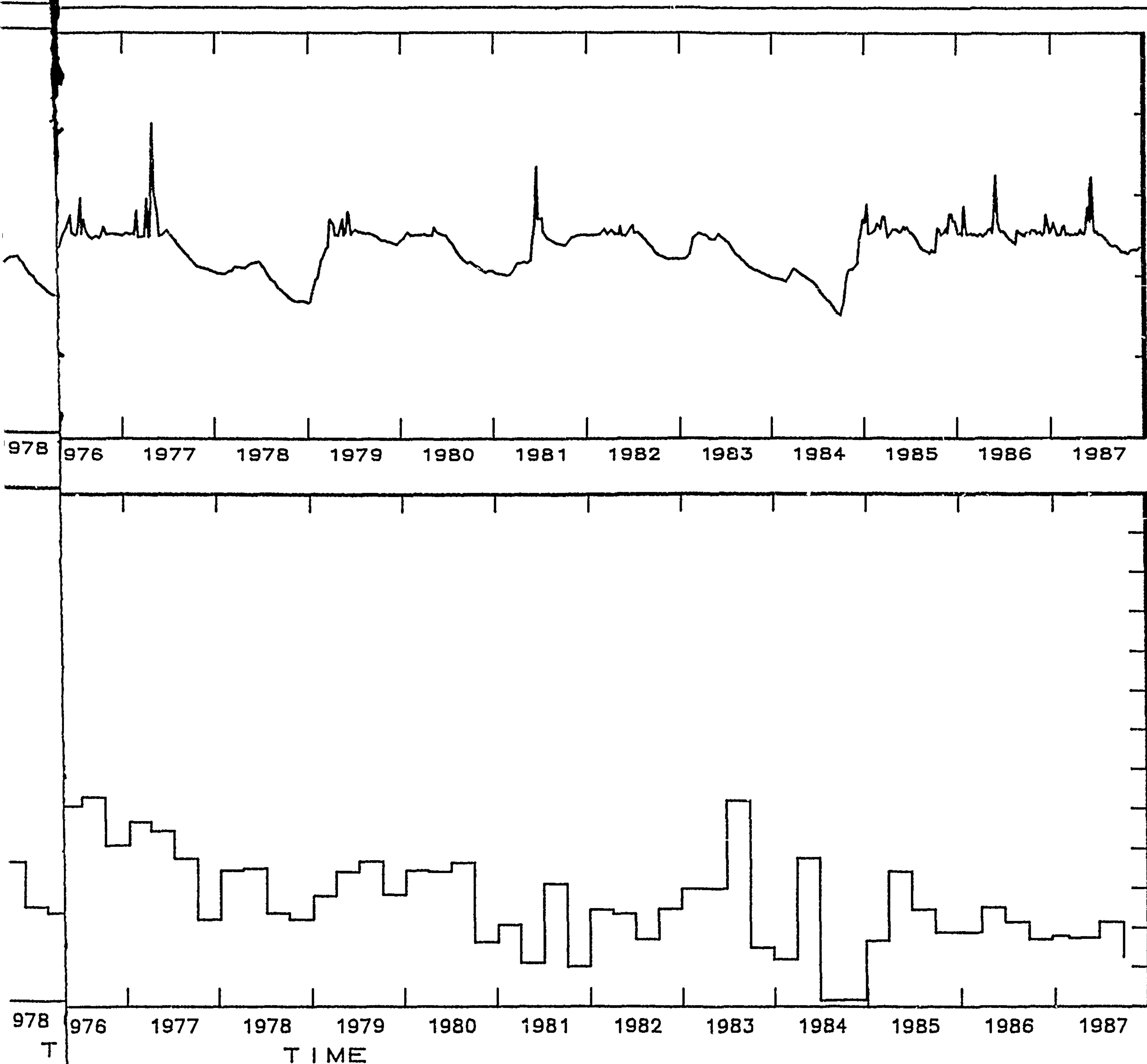
NTS

ETE
ER
ETE
MBE

WACO DAM
BOSQUE RIVER, TEXAS

LOCATION OF SPILLWAY PIEZOMETERS


LAKE ELEV IN FT - NGVD
480
470
460
450
440
430
1970
1971
1972
1973
1974
1975
1976
197
TOTAL GALLERY DRAINAGE FLOW-GPD
1400
1200
1000
800
600
400
200
1970
1971
1972
1973
1974
1975
1976
197
+ = NO READING



WACO LAKE
FORT WORTH DISTRICT
LAKE ELEVATION
AND
GALLERY DRAINAGE vs. TIME

PLATE 42

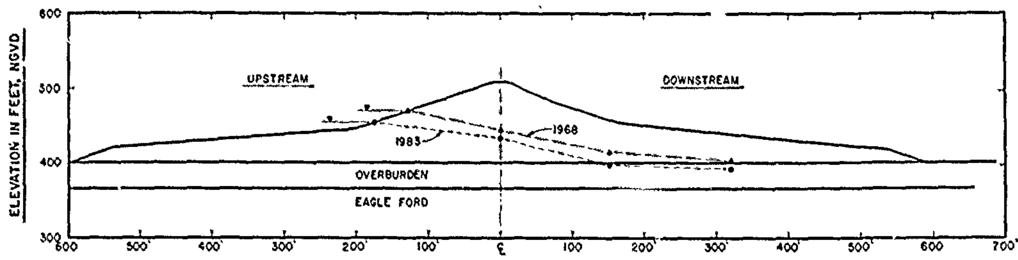


FIGURE 1

FILL-OVERBURDEN PIEZOMETERS
STATION 8+00

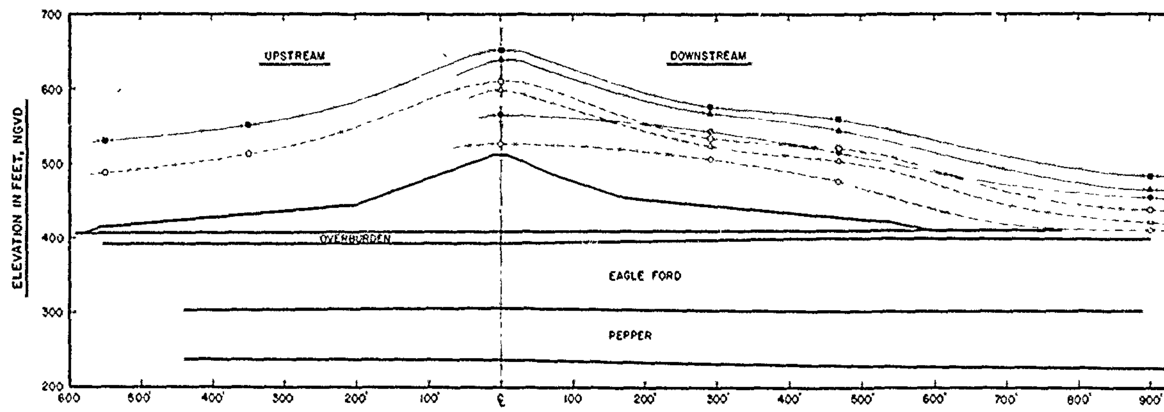


FIGURE 3

FOUNDATION PIEZOMETERS
STATION 44+00

LEGEND

- ● ○ 1964-65
- ▲ △ 1971
- ◆ ◇ 1983
- ■ PEPPER/DEL RIO
- ○ MID PEPPER

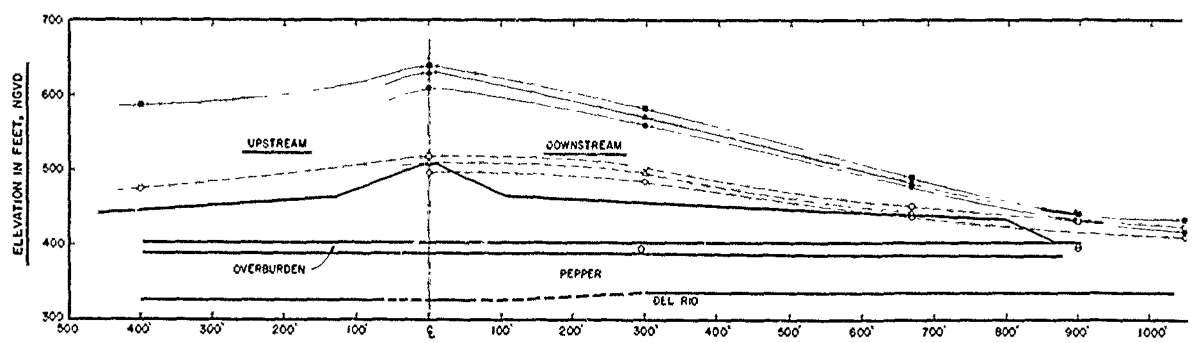
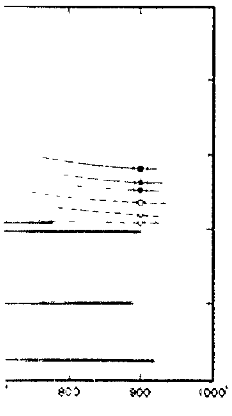


FIGURE 5

FOUNDATION PIEZOMETERS
STATION 55+00

LEGEND

- ● ○ 1964-65
- ▲ △ 1971
- ◆ ◇ 1983
- ■ PEPPER/DEL RIO
- ○ MID PEPPER (EL 370)



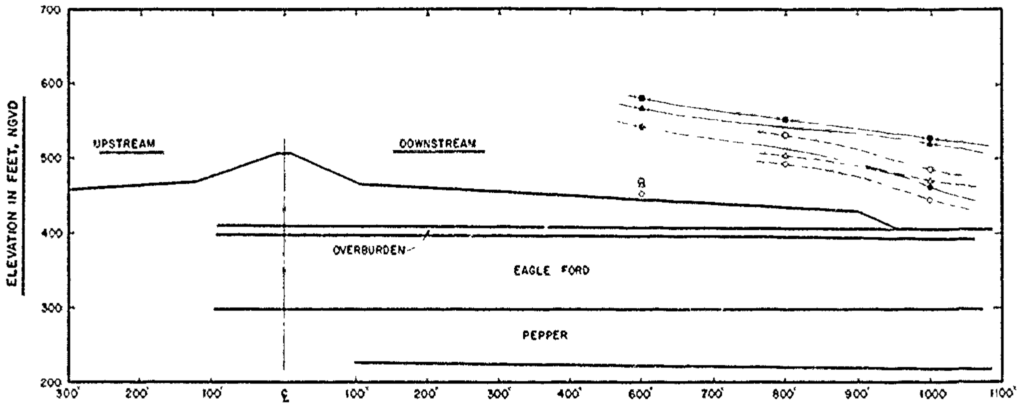
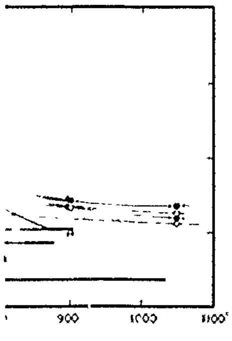


FIGURE 4

FOUNDATION PIEZOMETERS
STATION 51+00

LEGEND

- ● ○ 1964-65
- ▲ △ 1971
- ♦ ◇ 1983
- ■ PEPPER / DEL RIO
- ○ MID PEPPER



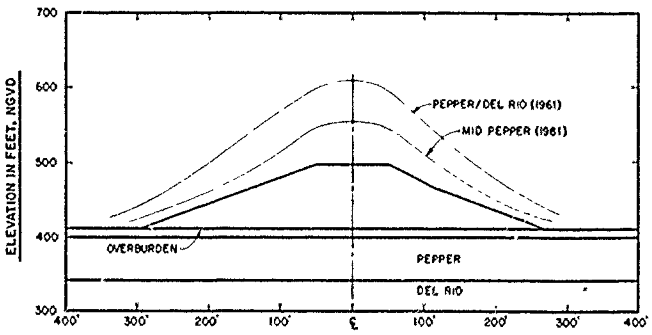


FIGURE 6

FOUNDATION PIEZOMETERS
STATION 55+00
(PRE-FAILURE)

SYMBOL NO.	ACTION	DATE	DESCRIPTION OF REVISION

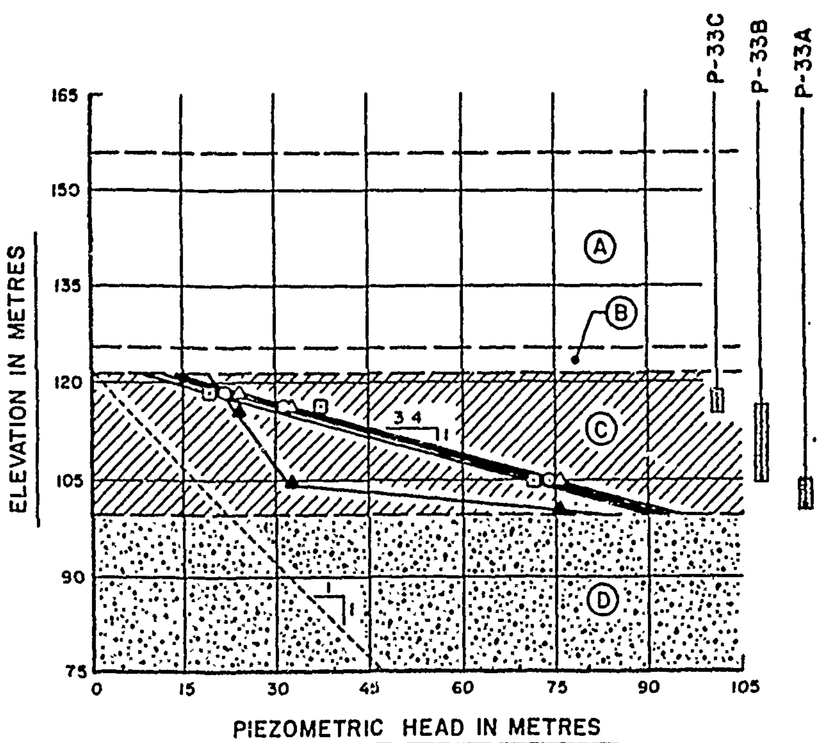
U.S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

DESIGNED BY
DRAWN BY
CHECKED BY

BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

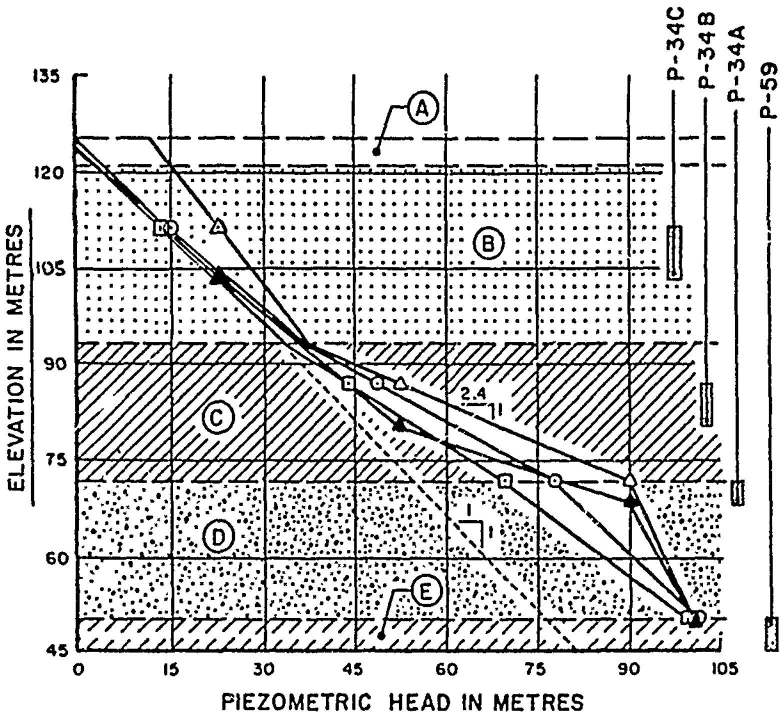
FILL-OVERBURDEN AND FOUNDATION PIEZOMETER PLOTS

SUBMITTED BY
ENGINEER.
INV NO
DATED
DRAWING NUMBER
SHEET NO
SEQUENCE NO.
CONTR. NO



△ 1963-65
⊙ 1971
⊡ 1983
▲ 1963-65 PRESSURE ASSUMED AT WELLPOINT
Ⓐ EMBANKMENT FILL
Ⓑ OVERBURDEN
Ⓒ PEPPER SHALE
Ⓓ DEL RIO SHALE

PORE PRESSURE GRADIENT
STATION 55+00, ℄

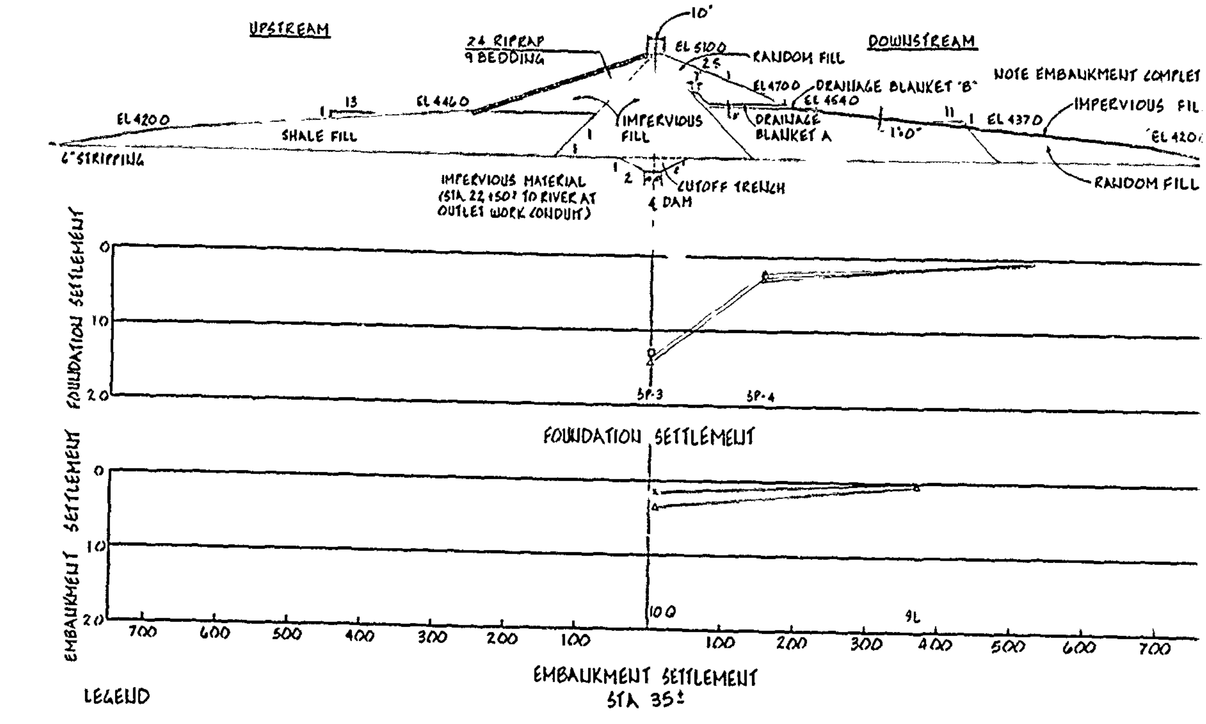


△ 1963-65
⊙ 1971
⊡ 1983
▲ 1963-65 PRESSURE ASSUMED AT WELLPOINT
Ⓐ OVERBURDEN
Ⓑ EAGLE FORD SHALE
Ⓒ PEPPER SHALE
Ⓓ DEL RIO SHALE
Ⓔ GEORGETOWN LIMESTONE

PORE PRESSURE GRADIENT

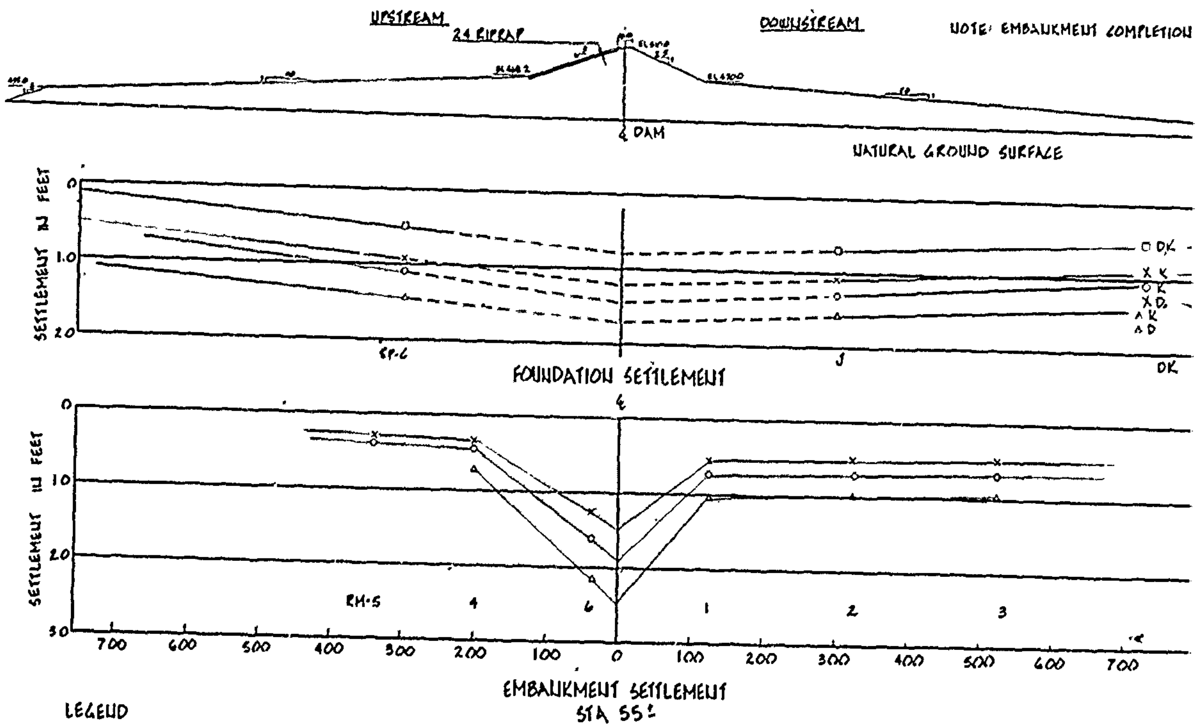
STATION 52+00, 275m D/S

WACO DAM BOSQUE RIVER, TEXAS PORE PRESSURE GRADIENT PLOTS
U.S. ARMY ENGINEER DISTRICT, FT. WORTH
PLATE



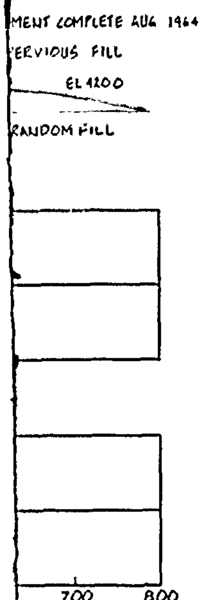
UPSTREAM
DOWNSTREAM
24 RIPRAP
9 BEDDING
EL 5100
RANDOM FILL
EL 4700
DRAINAGE BLANKET "B"
EL 4640
DRAINAGE BLANKET A
NOTE EMBANKMENT COMPLET
IMPERVIOUS FIL
EL 4370
EL 4460
EL 4200
SHALE FILL
IMPERVIOUS FILL
6" STRIPPING
IMPERVIOUS MATERIAL (STA 22+50± TO RIVER AT OUTLET WORK CONDUIT)
CUTOFF TRENCH
℄ DAM
RANDOM FILL
FOUNDATION SETTLEMENT
SP-3
SP-4
FOUNDATION SETTLEMENT
EMBANKMENT SETTLEMENT
700 600 500 400 300 200 100 100 200 300 400 500 600 700
EMBANKMENT SETTLEMENT
STA 35±
LEGEND
1966
1968
1971
1983

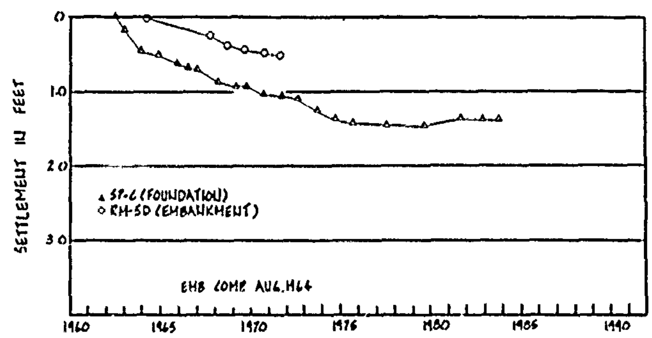
FIG. 1



UPSTREAM
24 RIPRAP
DOWNSTREAM
NOTE: EMBANKMENT COMPLETION
℄ DAM
NATURAL GROUND SURFACE
SETTLEMENT IN FEET
SP-6
DK
FOUNDATION SETTLEMENT
℄
SETTLEMENT IN FEET
RM-5
700 600 500 400 300 200 100 0 100 200 300 400 500 600 700
EMBANKMENT SETTLEMENT
STA 55±
LEGEND
1964
1968
1971
1983

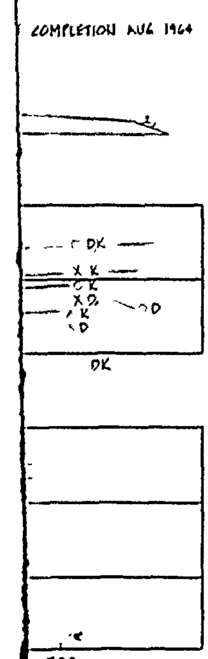
FIG. 2

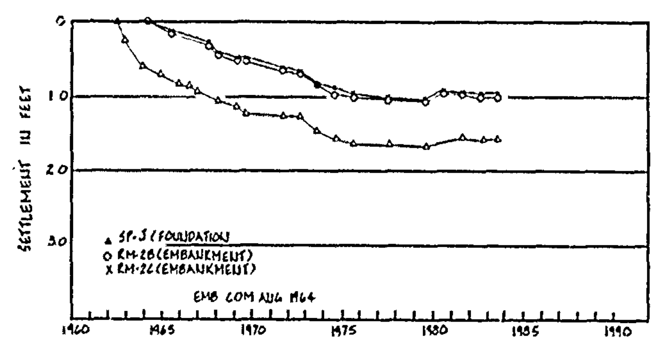




EMBANKMENT AND FOUNDATION
SETTLEMENT
STA 55+00, 300US

FIG. 3





EMBANKMENT AND FOUNDATION
SETTLEMENT
STA 55+00, 300DS

FIG. 4

NOTES:

SETTLEMENT PLATE D IS SET ON TOP OF SHALE, EL. 395.7,
SETTLEMENT PLATE K IS SET ON OVERBURDEN, EL. 408.0

U.S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

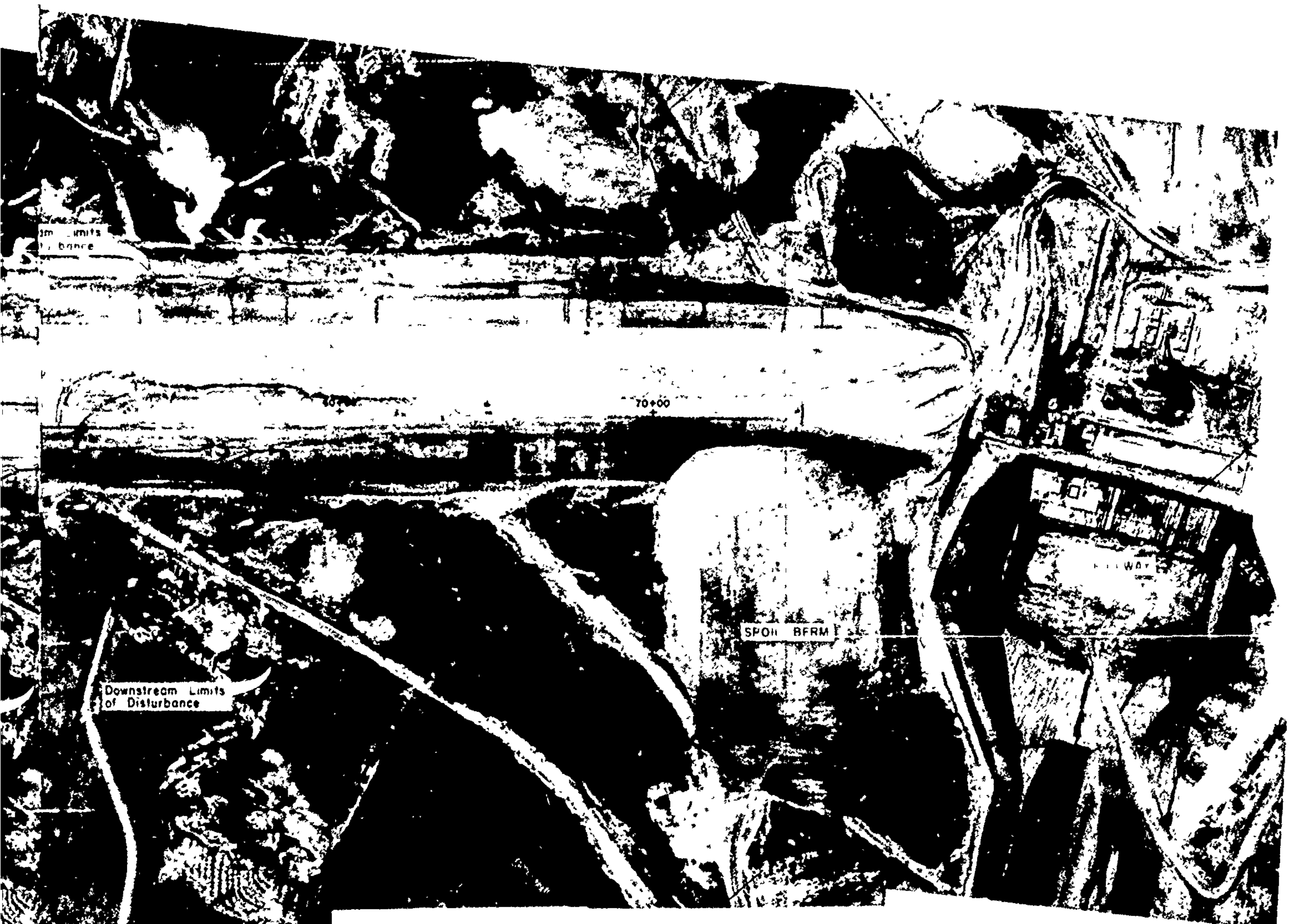
DESIGNED BY:
DRAWN BY:
REVIEWED BY:
SUBMITTED BY:
ENGINEER:

WACO DAM
BOSQUE RIVER, TEXAS

EMBANKMENT AND FOUNDATION
SETTLEMENT PLOTS

INV. NO.
DATED:
CONTR. NO
DRAWING NUMBER
SHEET NO. of
SEQUENCE NO.


N
Upstream
Upstream Limits
of Disturbance
40+00
50+00
OUTLET
WORKS
Downstream



APPROX SCALE OF FEET

200 0 200 400

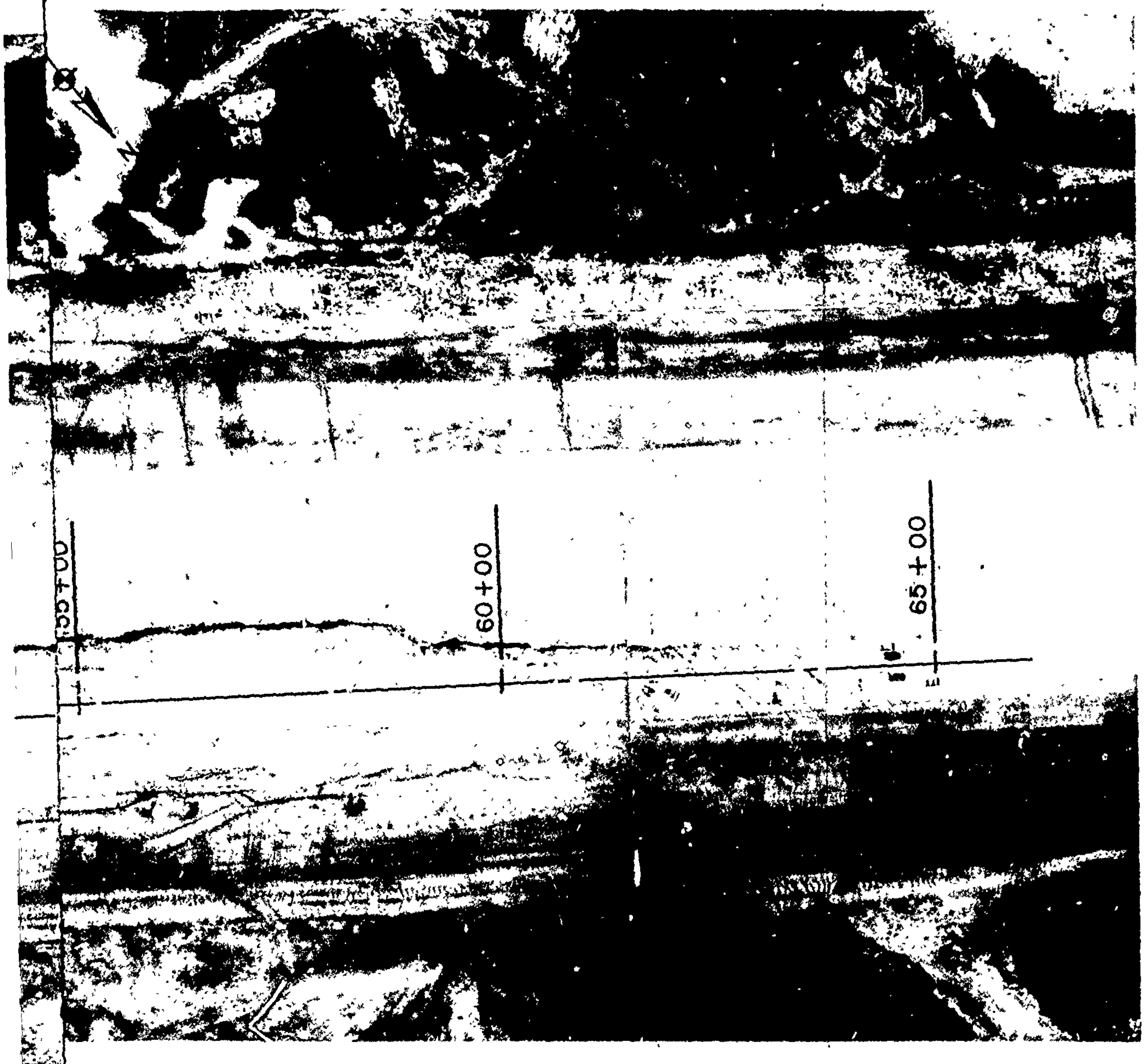
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

AERIAL VIEW OF DAM

IN 2 SHEETS	SCALE AS SHOWN	SHEET NO 1
U.S. ARMY ENGINEER DISTRICT, FORT WORTH		JAN. 1963

CORPS OF ENGINEERS

F



PLAN

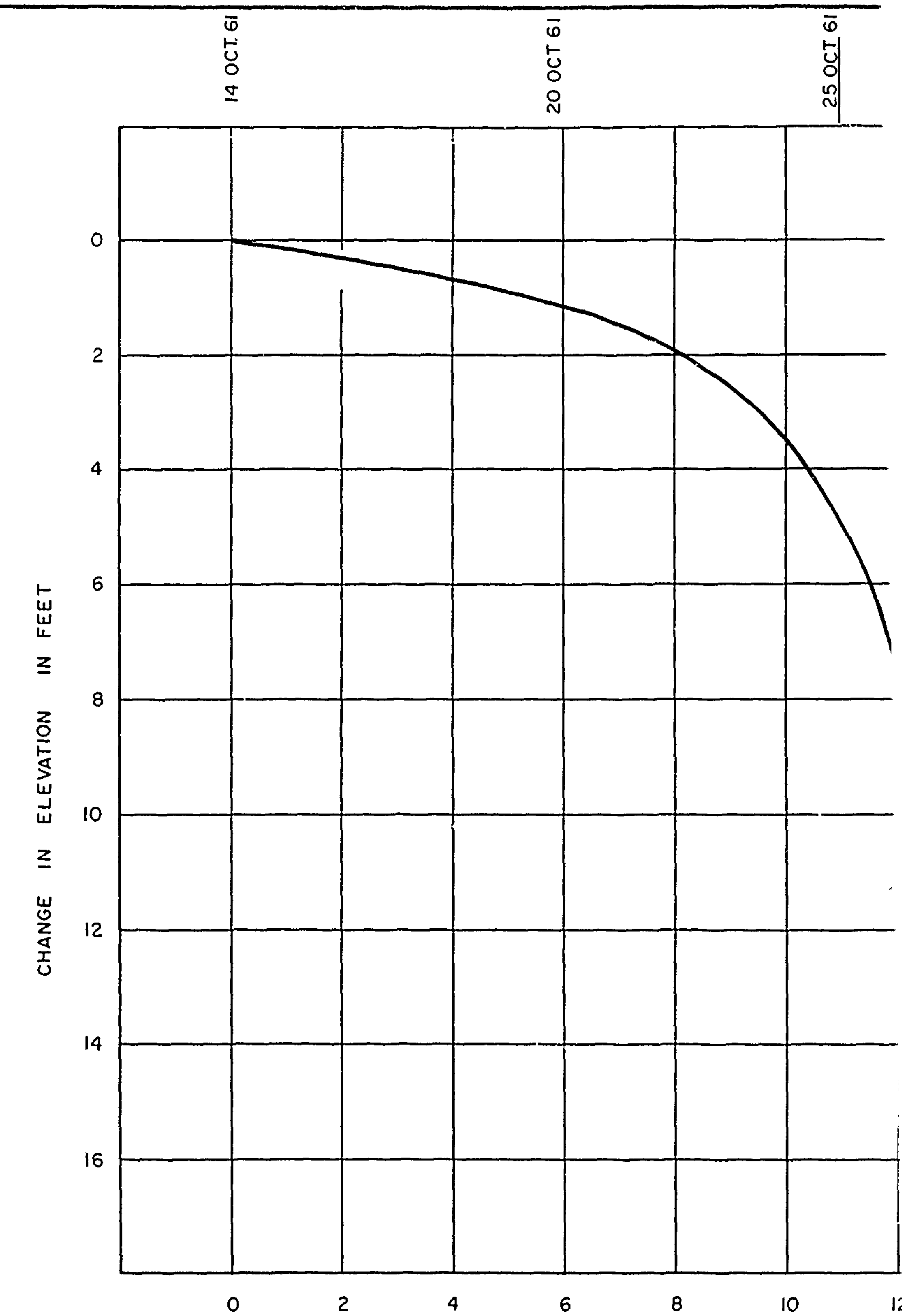
N.T.S.

WACO DAM

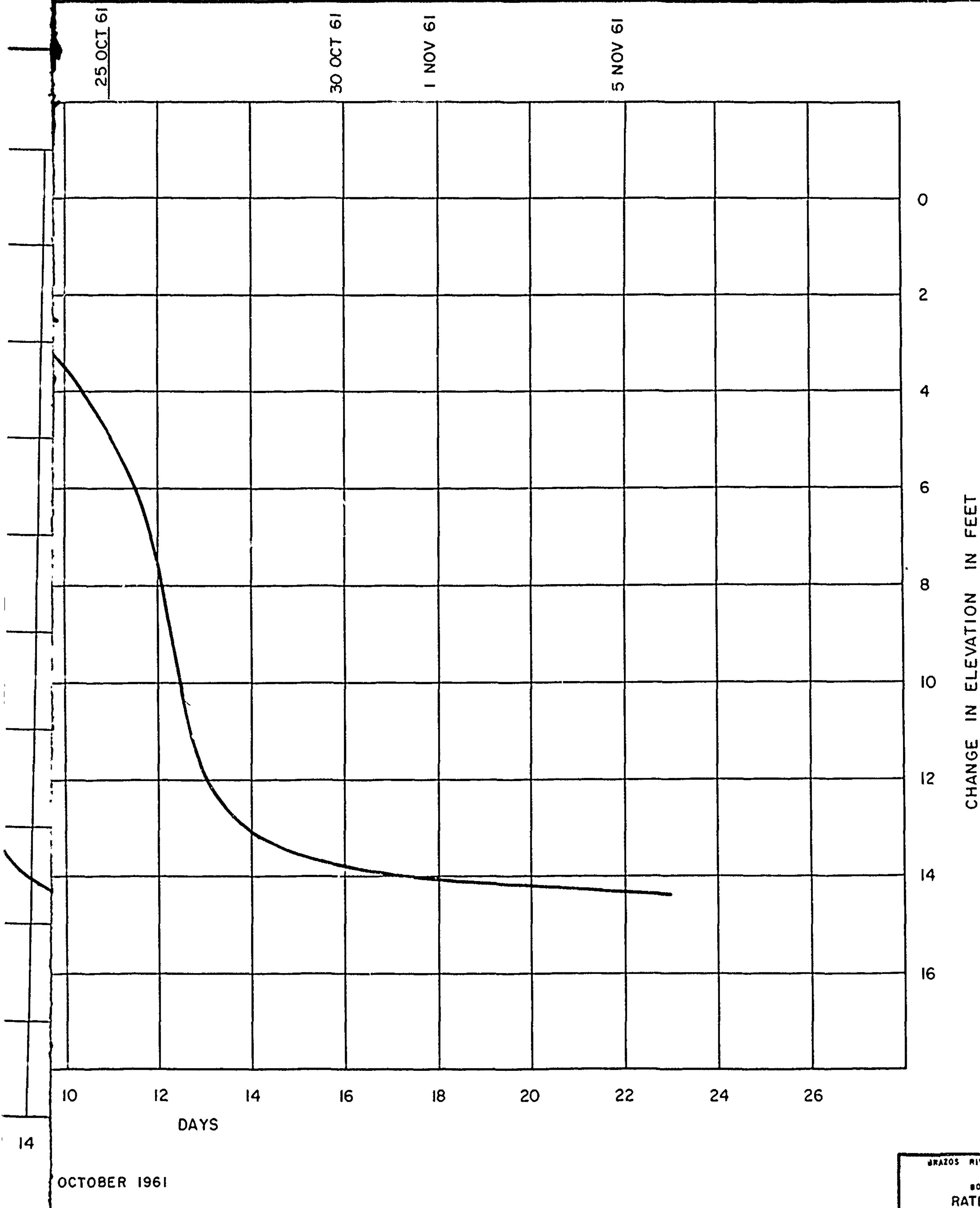
AERIAL VIEW OF DAM
SLIDE AREA

U.S. ARMY ENGINEER DISTRICT, FT. WORTH

CORPS OF ENGINEERS

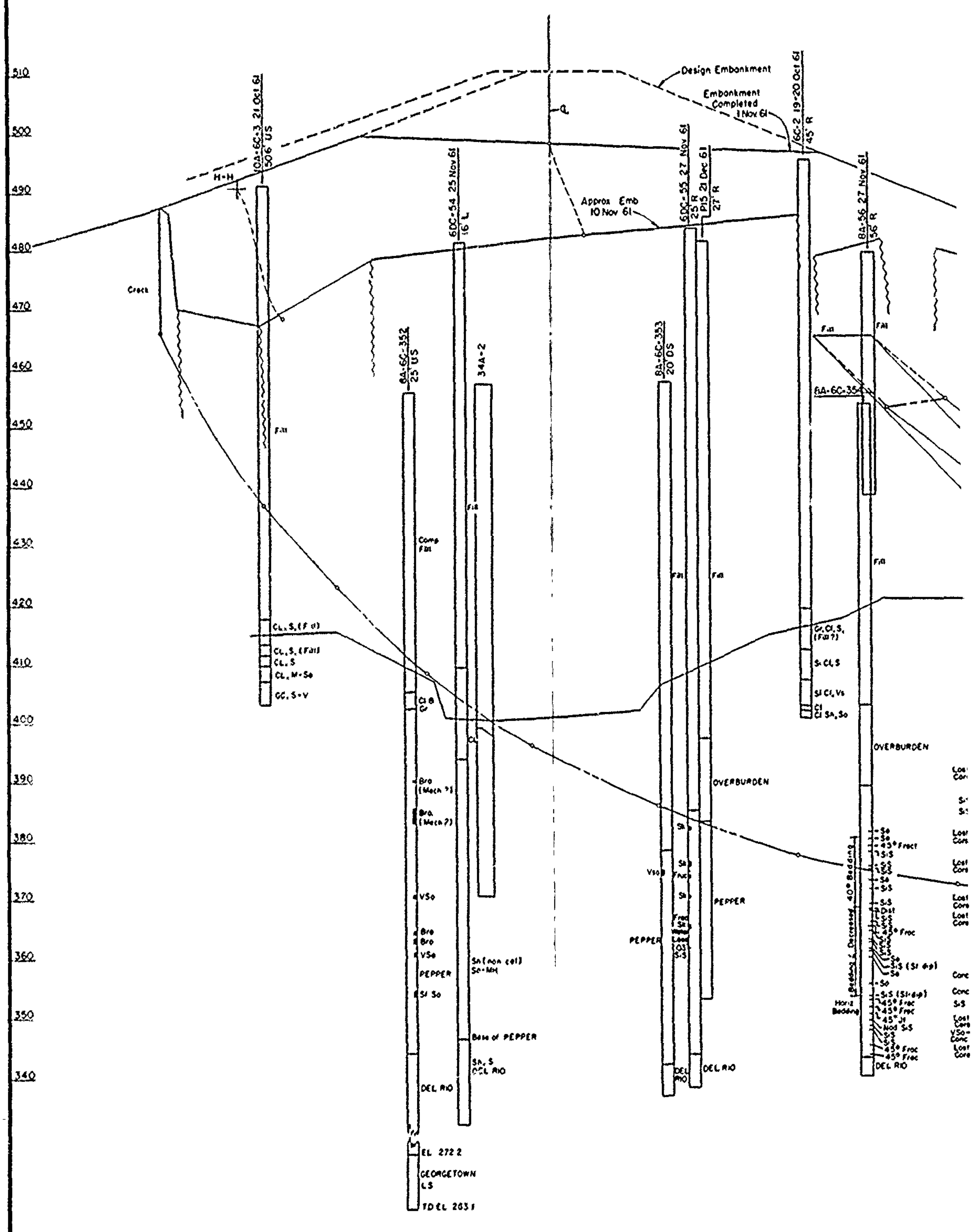


NOTE:
MOVEMENT PRIOR TO 14 OCTOBER 196
WAS NOT MEASURED.

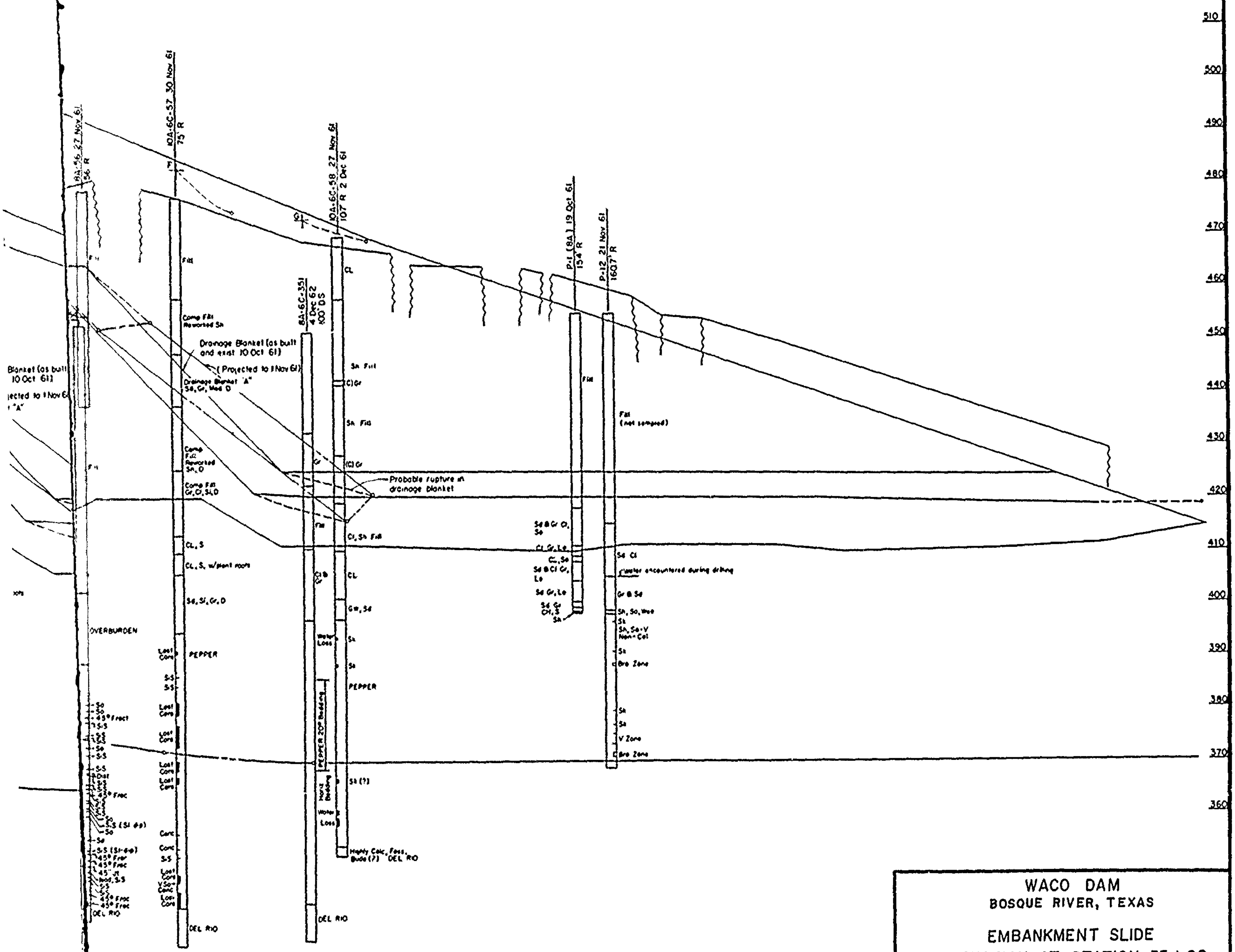


BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS
RATE OF MOVEMENT
CENTERLINE POINT
STATION 55+00
EMBANKMENT
SCALES AS SHOWN
U S ARMY ENGINEER DISTRICT, FORT WORTH JAN 1

CORPS OF ENGINEERS



U.S. ARMY

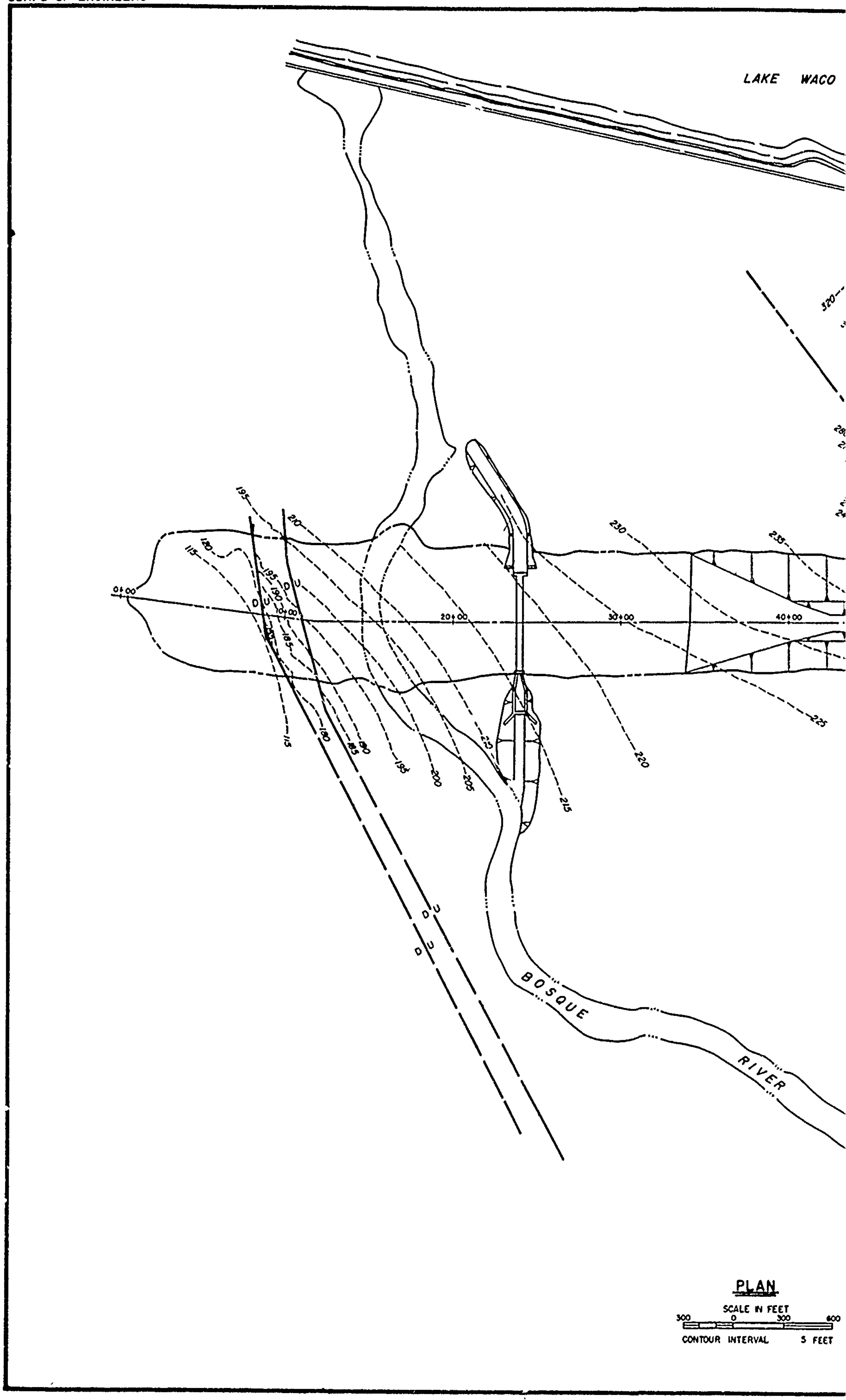


WACO DAM
BOSQUE RIVER, TEXAS

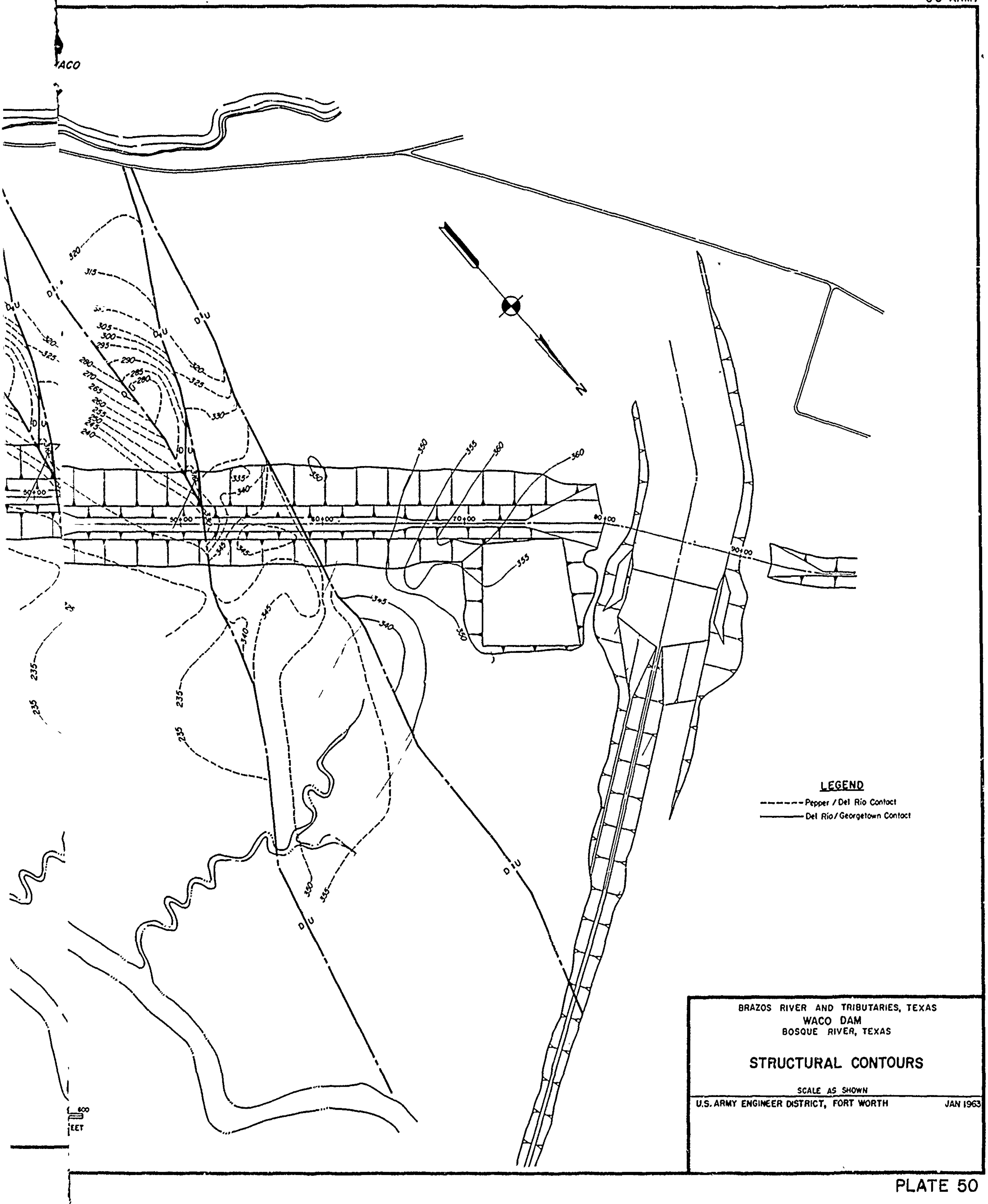
EMBANKMENT SLIDE
SECTION AT STATION 55+00

PLATE 49

CORPS OF ENGINEERS



US ARMY



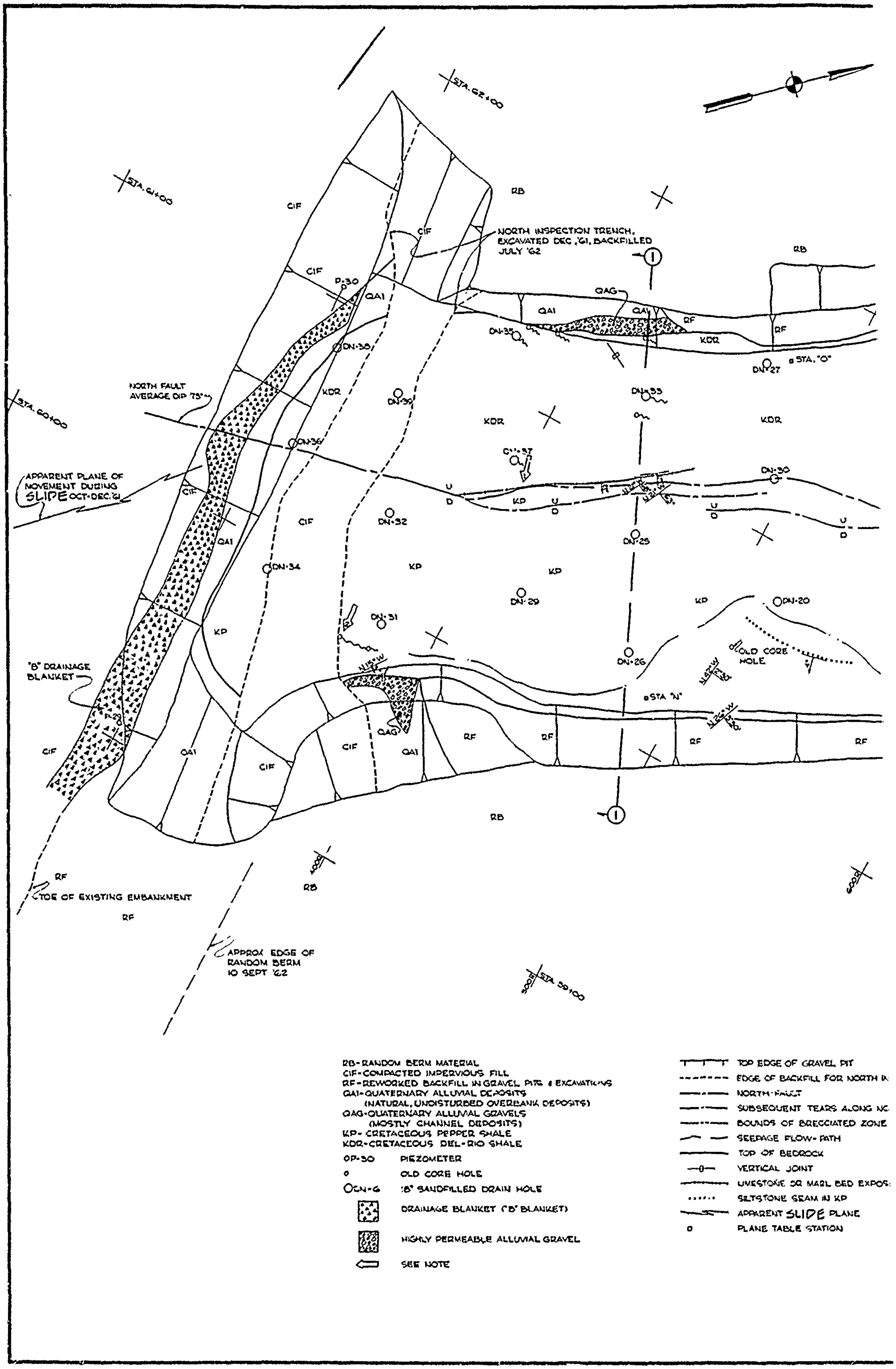
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

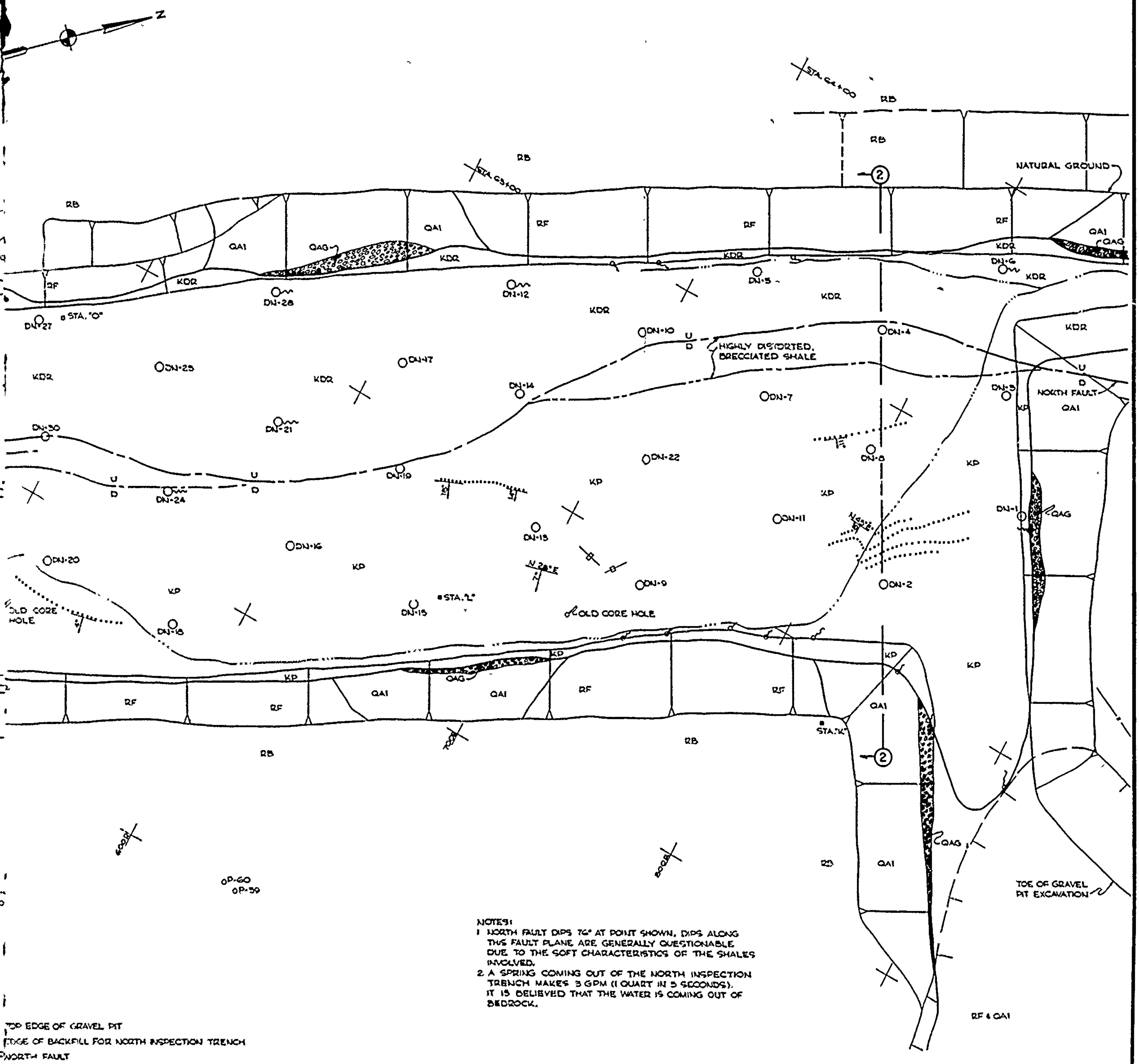
STRUCTURAL CONTOURS

SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN 1963

PLATE 50


STA. 62+00
STA. 61+00
STA. 60+00
STA. 59+00
NORTH INSPECTION TRENCH, EXCAVATED DEC. '61, BACKFILLED JULY '62
NORTH FAULT AVERAGE DIP 73°
APPARENT PLANE OF MOVEMENT DURING SLIDE OCT-DEC. '61
"B" DRAINAGE BLANKET
TOE OF EXISTING EMBANKMENT
APPROX EDGE OF RANDOM BERM 10 SEPT '62
OLD CORE HOLE
STA. "O"
STA. "N"
RB
CIF
RF
QAI
QAG
KP
KDR
P-30
DN-38
DN-39
DN-36
DN-35
DN-33
DN-27
DN-30
DN-32
DN-34
DN-25
DN-29
DN-31
DN-20
DN-26
RB-RANDOM BERM MATERIAL
CIF-COMPACTED IMPERVIOUS FILL
RF-REWORKED BACKFILL IN GRAVEL PITS & EXCAVATIONS
QAI-QUATERNARY ALLUVIAL DEPOSITS
(NATURAL, UNDISTURBED OVERBANK DEPOSITS)
QAG-QUATERNARY ALLUVIAL GRAVELS
(MOSTLY CHANNEL DEPOSITS)
KP-CRETACEOUS PEPPER SHALE
KDR-CRETACEOUS DEL-RIO SHALE
OP-30 PIEZOMETER
OLD CORE HOLE
ODN-6 18" SANDFILLED DRAIN HOLE
DRAINAGE BLANKET ("B" BLANKET)
HIGHLY PERMEABLE ALLUVIAL GRAVEL
SEE NOTE
TOP EDGE OF GRAVEL PIT
EDGE OF BACKFILL FOR NORTH I
NORTH FAULT
SUBSEQUENT TEARS ALONG N
BOUNDS OF BRECCIATED ZONE
SEEPAGE FLOW-PATH
TOP OF BEDROCK
VERTICAL JOINT
LIMESTONE OR MARL BED EXPOS
SILTSTONE SEAM IN KP
APPARENT SLIDE PLANE
PLANE TABLE STATION



NOTES:
1. NORTH FAULT DIPS 76° AT POINT SHOWN. DIPS ALONG THIS FAULT PLANE ARE GENERALLY QUESTIONABLE DUE TO THE SOFT CHARACTERISTICS OF THE SHALES INVOLVED.
2. A SPRING COMING OUT OF THE NORTH INSPECTION TRENCH MAKES 3 GPM (1 QUART IN 5 SECONDS). IT IS BELIEVED THAT THE WATER IS COMING OUT OF BEDROCK.

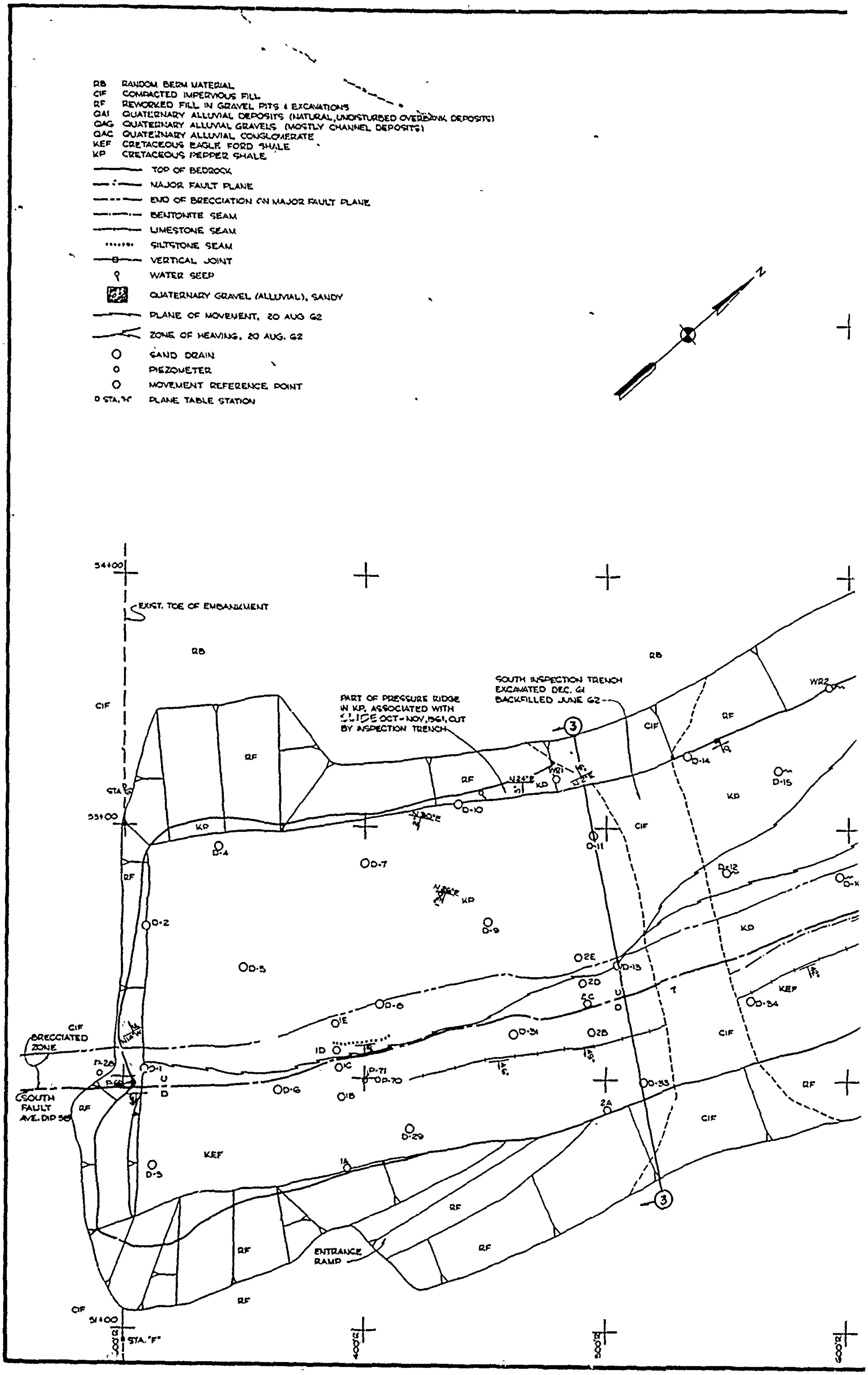
TOP EDGE OF GRAVEL PIT
EDGE OF BACKFILL FOR NORTH INSPECTION TRENCH
NORTH FAULT
SUBSEQUENT TEARS ALONG NORTH FAULT
BOUNDS OF BRECCIATED ZONE
SEEPAGE FLOW-PATH
TOP OF BEDROCK
VERTICAL JOINT
LIMESTONE OR MARL BED EXPOSED IN KDR
SILTSTONE SEAM IN KP
APPARENT SLIDE PLANE
PLANE TABLE STATION

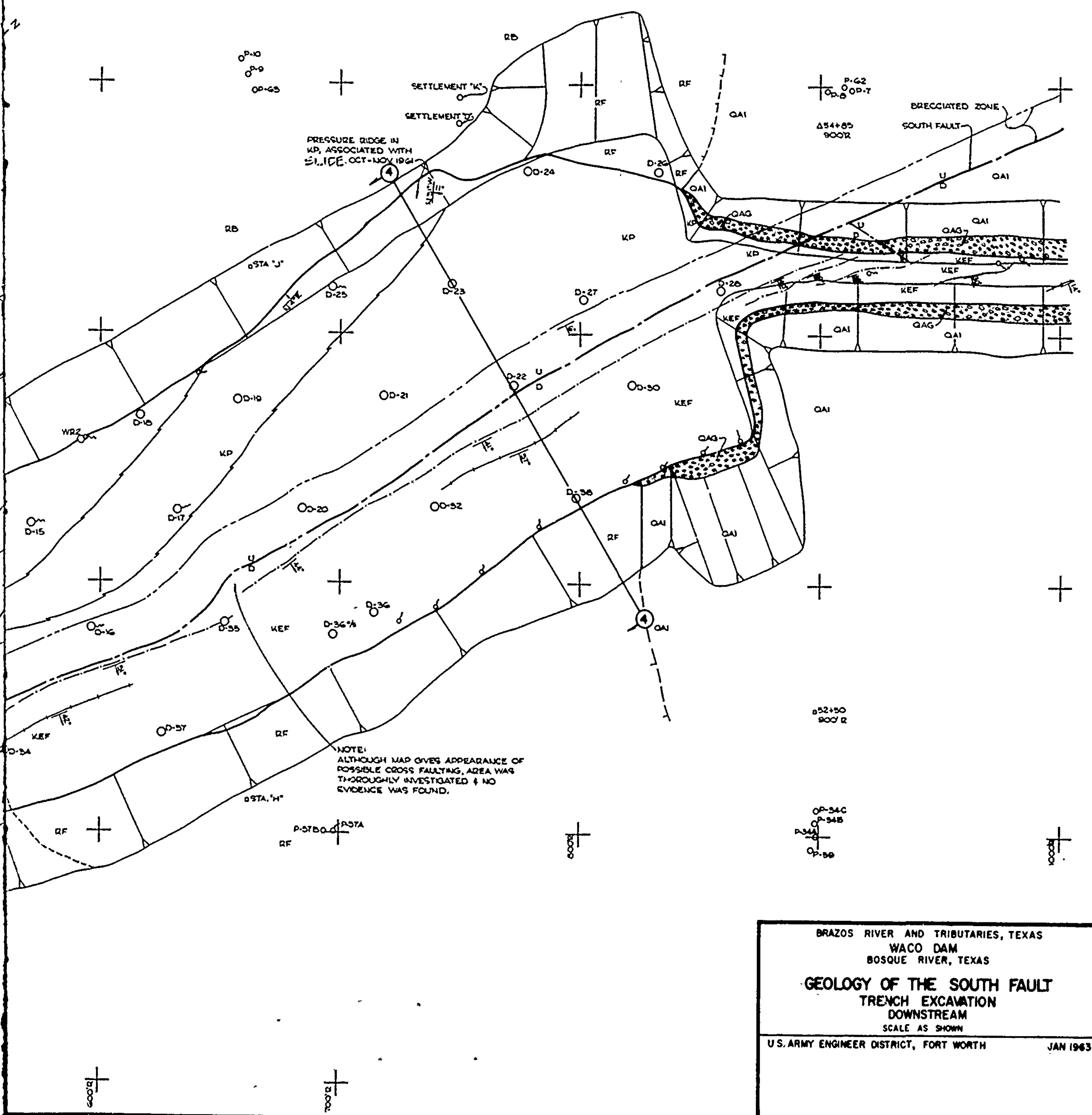
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

GEOLOGY OF THE NORTH FAULT
TRENCH EXCAVATION
DOWNSTREAM
SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JUNE 1969

PLATE 51


RB RANDOM BERM MATERIAL
CIF COMPACTED IMPERVIOUS FILL
RF REWORKED FILL IN GRAVEL PITS & EXCAVATIONS
QAI QUATERNARY ALLUVIAL DEPOSITS (NATURAL, UNDISTURBED OVERBANK DEPOSITS)
QAG QUATERNARY ALLUVIAL GRAVELS (MOSTLY CHANNEL DEPOSITS)
QAC QUATERNARY ALLUVIAL CONGLOMERATE
KEF CRETACEOUS EAGLE FORD SHALE
KP CRETACEOUS PEPPER SHALE
TOP OF BEDROCK
MAJOR FAULT PLANE
END OF BRECCIATION ON MAJOR FAULT PLANE
BENTONITE SEAM
LIMESTONE SEAM
SILTSTONE SEAM
VERTICAL JOINT
WATER SEEP
QUATERNARY GRAVEL (ALLUVIAL), SANDY
PLANE OF MOVEMENT, 20 AUG 62
ZONE OF HEAVING, 20 AUG. 62
SAND DRAIN
PIEZOMETER
MOVEMENT REFERENCE POINT
PLANE TABLE STATION
N
54+00
EXIST. TOE OF EMBANKMENT
SOUTH INSPECTION TRENCH EXCAVATED DEC. 61 BACKFILLED JUNE 62
PART OF PRESSURE RIDGE IN KP, ASSOCIATED WITH SLIDE OCT-NOV, 1961, CUT BY INSPECTION TRENCH
53+00
BRECCIATED ZONE
SOUTH FAULT AVE. DIP 58°
ENTRANCE RAMP
51+00
STA. "F"
300'L
400'L
500'L
600'L

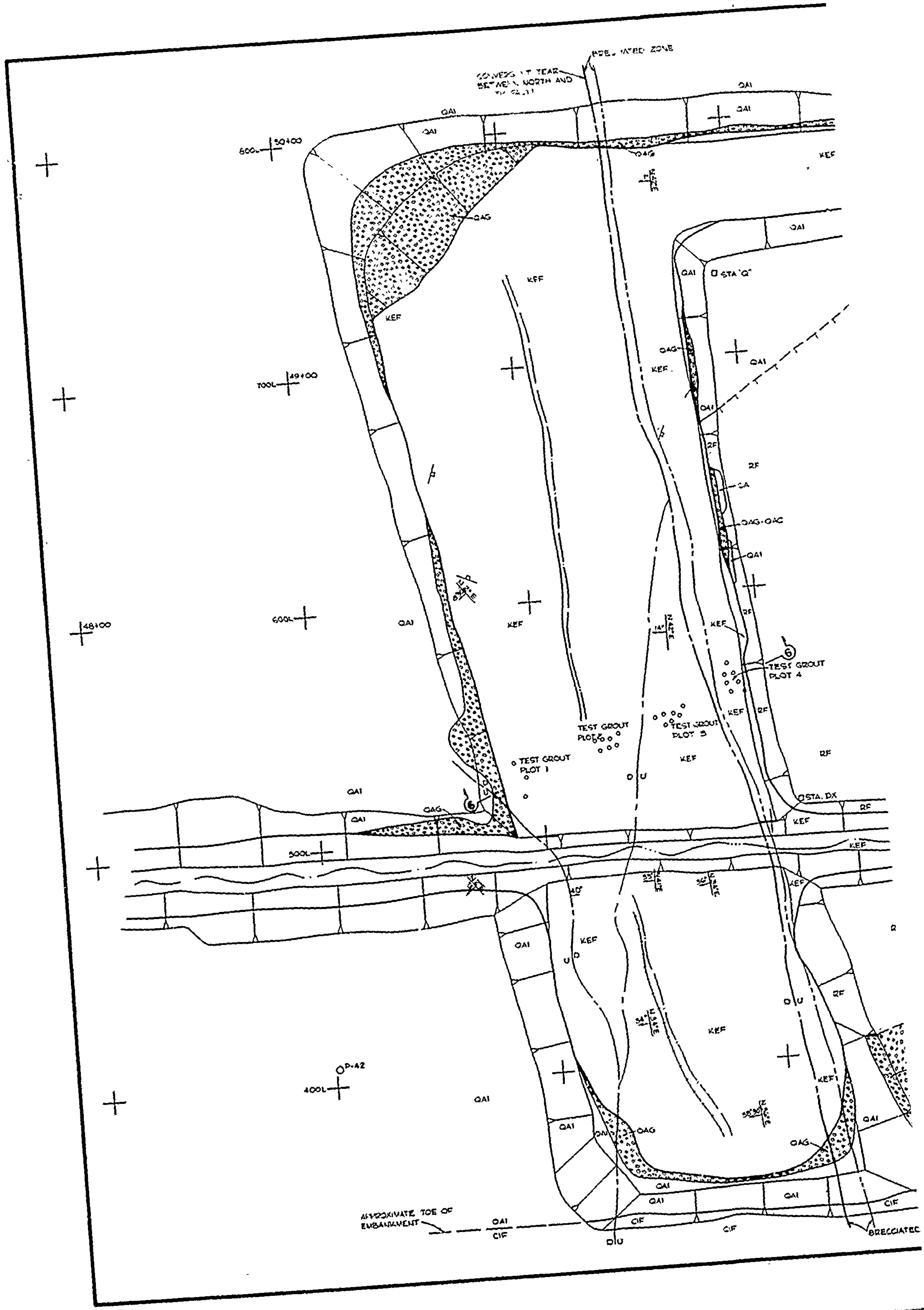


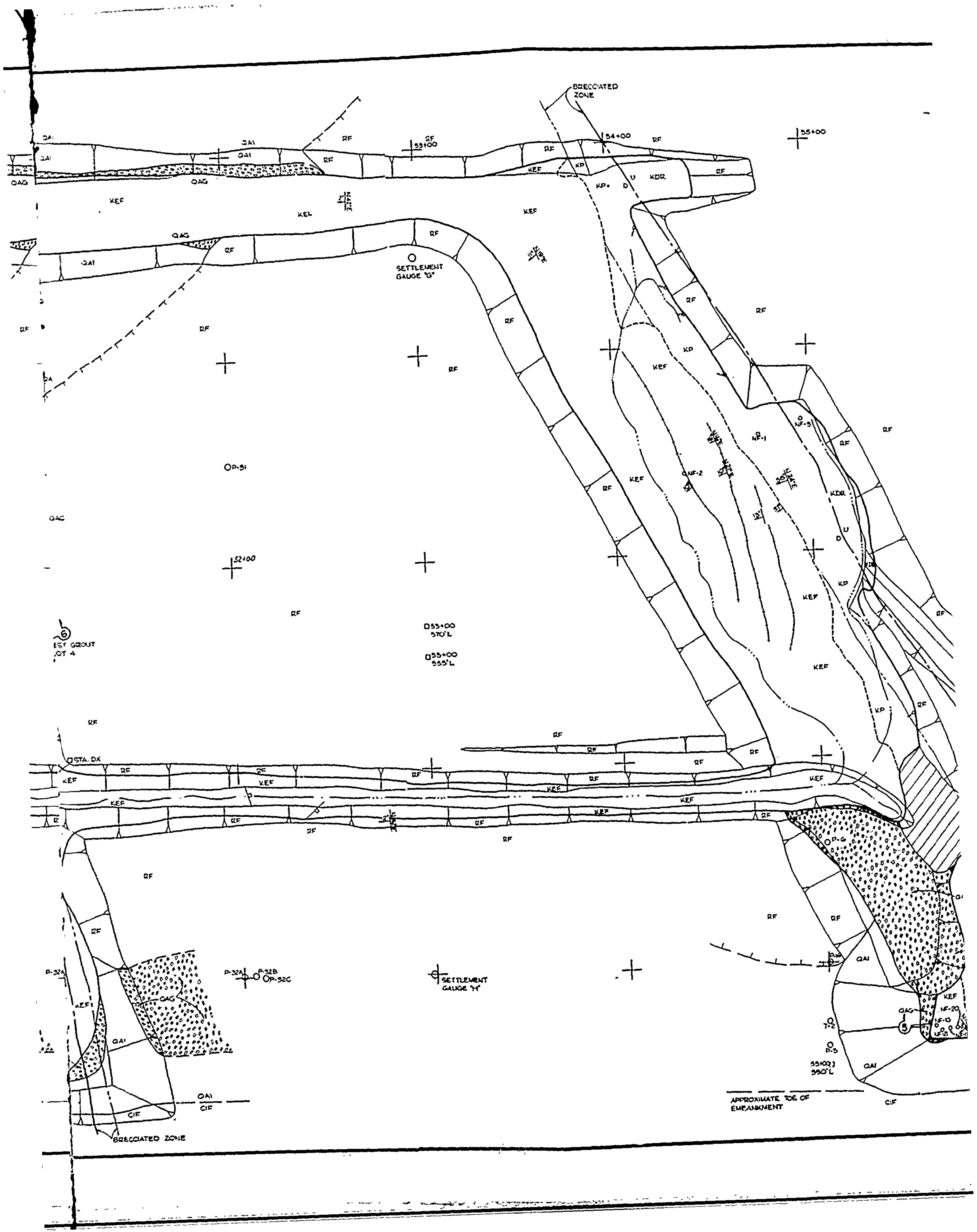
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

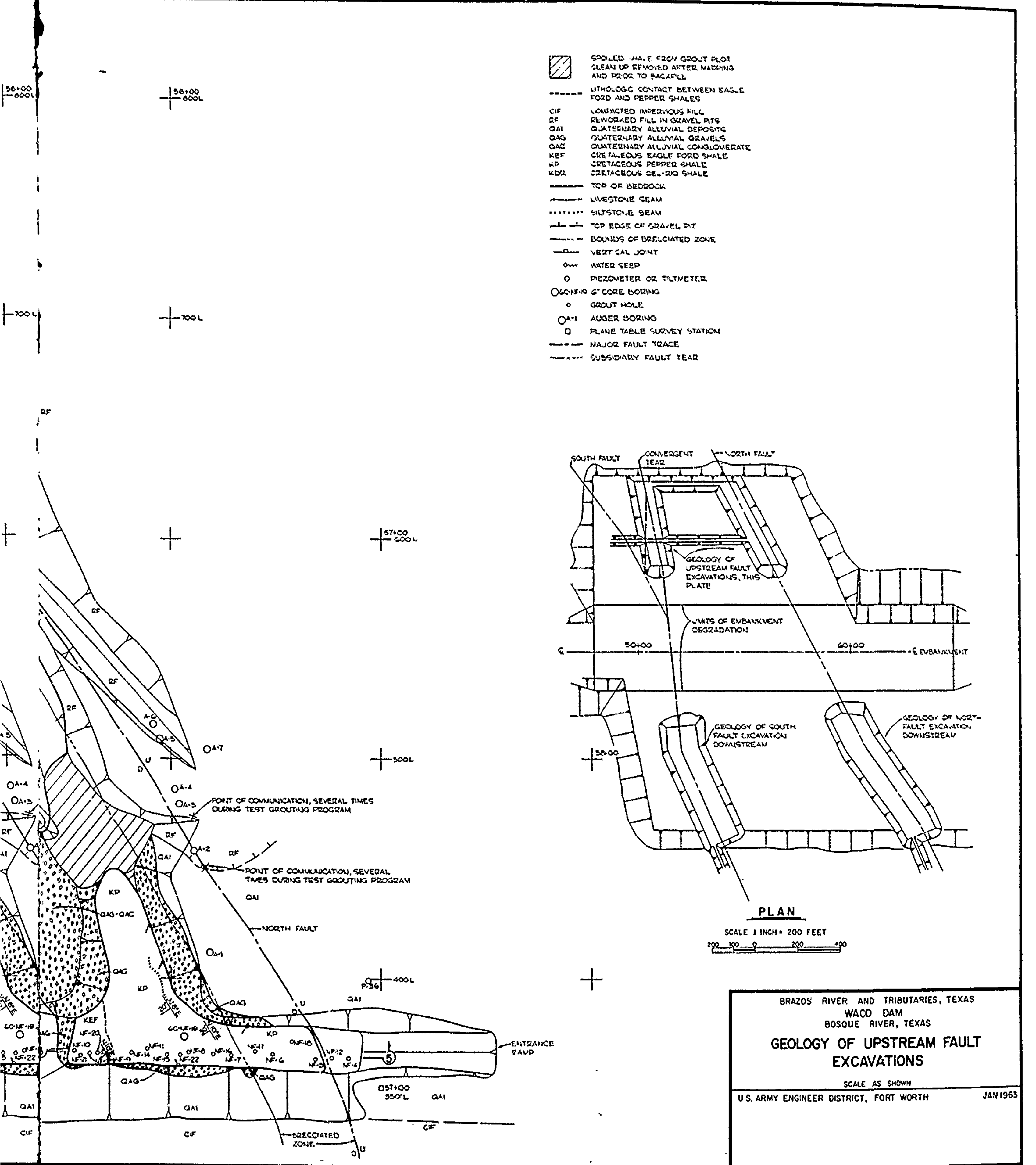
GEOLOGY OF THE SOUTH FAULT
TRENCH EXCAVATION
DOWNSTREAM

SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN 1963


BRECCIATED ZONE
CONVERGENT TEAR BETWEEN NORTH AND
QAI
QAG
KEF
STA "Q"
RF
CA
QAG-QAC
TEST GROUT PLOT 4
TEST GROUT PLOT 2
TEST GROUT PLOT 3
TEST GROUT PLOT 1
STA. DX
600L
50+00
700L
49+00
48+00
500L
400L
P-42
APPROXIMATE TOE OF EMBANKMENT
CIF
D U
BRECCIATED


BRECCIATED ZONE
54+00
55+00
53+00
SETTLEMENT GAUGE "G"
52+00
D53+00 570'L
D53+00 555'L
EST GROUT LOT 4
STA. DX
SETTLEMENT GAUGE "H"
P-32A
P-32B
P-32C
OP-31
NF-1
NF-2
NF-3
P-6
P-5
7-2
53+00 550'L
APPROXIMATE TOE OF EMBANKMENT
BRECCIATED ZONE
QAI
CIF

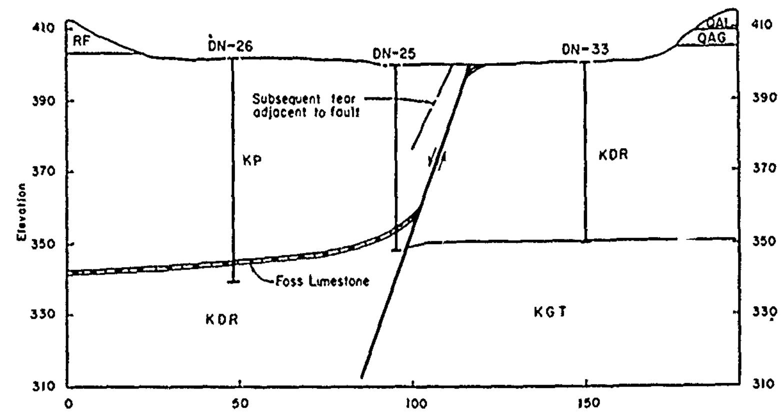


BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

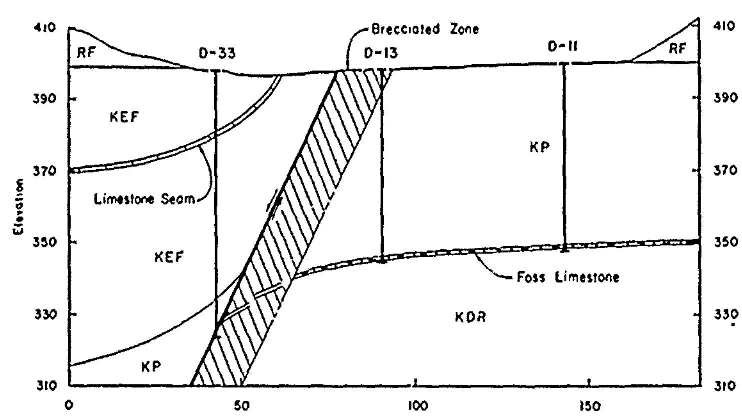
GEOLOGY OF UPSTREAM FAULT EXCAVATIONS

SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH JAN 1963



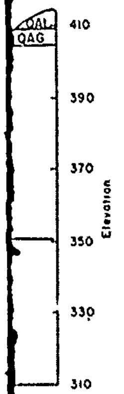
SECTION 1-1
NORTH FAULT

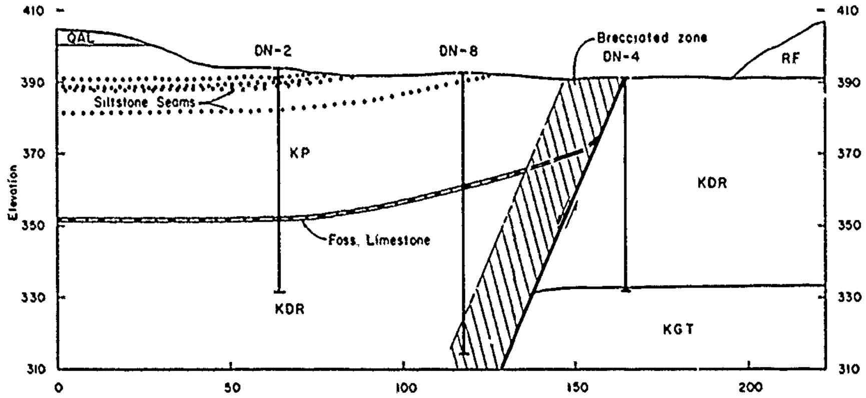


SECTION 3-3
SOUTH FAULT

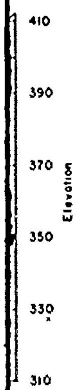
LEGEND

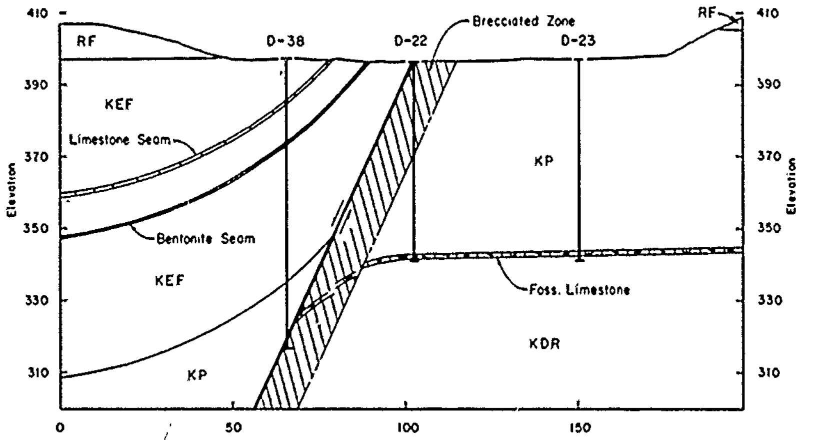
RF	Random Fill Material
QAL	Quaternary Alluvial Deposits (Undisturbed overbank deposits)
QAG	Quaternary Alluvial Gravels (Undisturbed channel deposits)
KEF	Cretaceous Eagle Ford shale
KP	Cretaceous Pepper shale
KDR	Cretaceous Del Rio shale
KGT	Cretaceous Georgetown Limestone





SECTION 2-2
NORTH FAULT





SECTION 4-4
SOUTH FAULT

LEGEND

...terial
...vial Deposits
...d overbank deposits)
...vial Gravels
...d channel deposits)
...e Ford shale
...per shale
... Rio shale
...rgetown Limestone

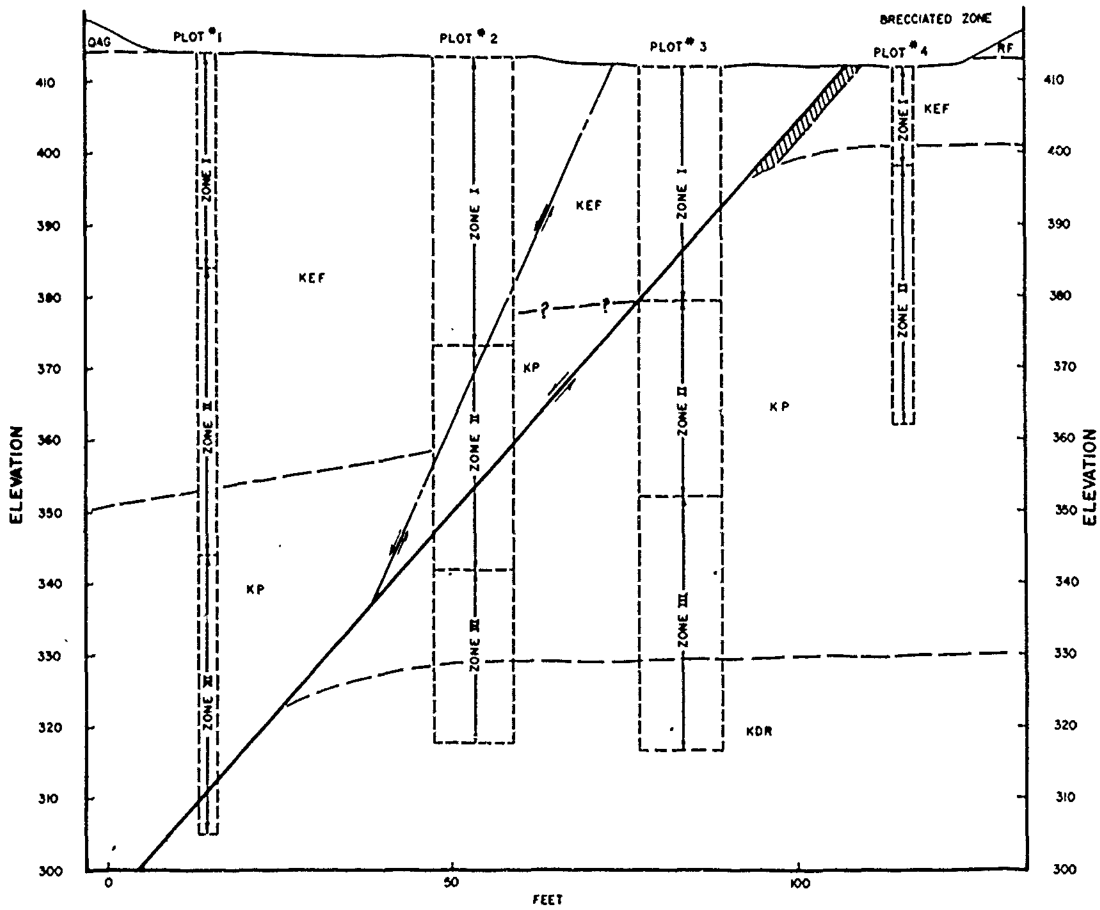
Geologic contact
Limestone seam (in KEF)
Bentonite seam (in KEF)
Siltstone seam (in KP)
Foss limestone (marks KP/KDR contact)
Fault
Limits of brecciated zone (adjacent to faults)
DN-4 Location of 18" diameter sand drain in section

BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

GEOLOGIC SECTIONS
DOWNSTREAM FAULT EXCAVATIONS

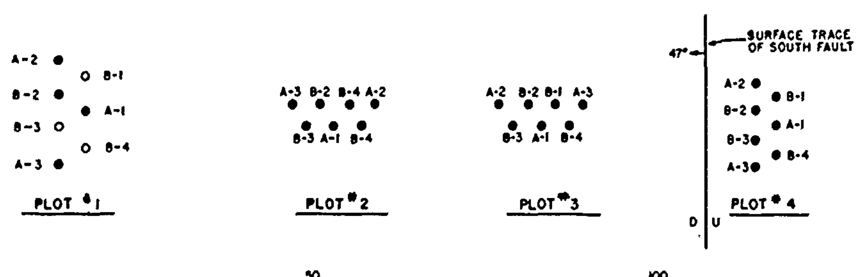
SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH JAN 1963

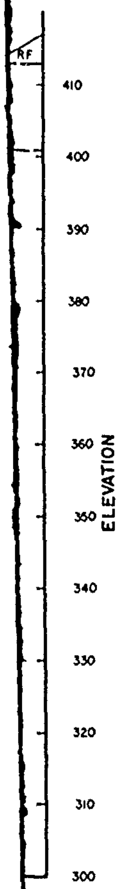


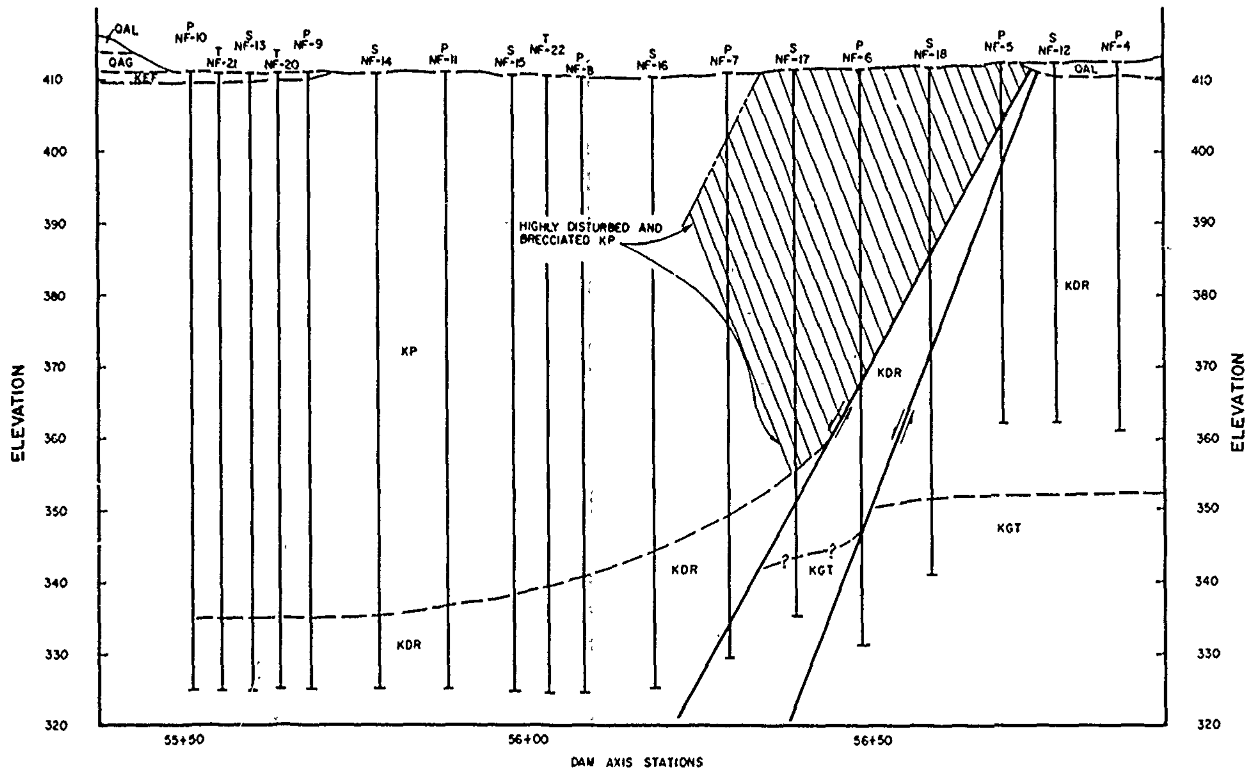
SECTION "6-6"

SOUTH FAULT-SECTION ON GROUT PLOTS



LAYOUT OF GROUT HOLES
SOUTH FAULT GROUT PLOTS





SECTION "5-5"

NORTH FAULT - GROUT LINE

LEGEND

GENERAL

RF	REWORKED FILL MATERIALS
QAL	QUATERNARY ALLUVIAL DEPOSITS (NATURAL UNDISTURBED OVERBANK DEPOSITS)
QAG	QUATERNARY ALLUVIAL GRAVELS (CHANNEL DEPOSITS)
KEF	CRETACEOUS EAGLE FORD SHALE
KP	CRETACEOUS PEPPER SHALE
KDR	CRETACEOUS DEL RIO SHALE
KGT	CRETACEOUS GEORGETOWN LIMESTONE
	GEOLOGIC CONTACT
	FAULT
	INFERRED FAULT
	LIMIT OF BRECCIATED OR DISTURBED ZONE

SECTION "5-5"

P	PRIMARY GROUT HOLE
S	SECONDARY GROUT HOLE
T	TERTIARY GROUT HOLE
	LOCATION OF GROUT HOLE IN SECTION

NOTE: THE TEST GROUT PROGRAM ON THE NORTH FAULT CONSISTS OF A SINGLE LINE OF GROUT HOLES DRILLED AND GROUTED BY SPLIT SPACING STOP-GROUTING METHODS

GROUT HOLE DATA IS SHOWN ON PLATE COMPILED GROUT HOLE DATA, NORTH FAULT, TEST GROUT PROGRAM

DAM AXIS STATIONS ARE NOT SHOWN ON THIS SECTION BECAUSE THE SECTION IS NOT PARALLEL TO DAM AXIS

SECTION "6-6"

A	PRIMARY GROUT HOLE
B	SECONDARY GROUT HOLE
	LIMITS OF GROUT PLOT IN SECTION
●	DRILLED AND GROUTED HOLE
O	PROPOSED BUT NOT DRILLED AND GROUTED HOLE

NOTE: THE TEST GROUT PROGRAM ON THE SOUTH FAULT CONSISTS OF 4 PLOTS OF 7 HOLES EACH. EACH PLOT WAS STAGE GROUTED IN AN A-B (PRIMARY-SECONDARY) SEQUENCE

GROUT HOLE DATA IS SHOWN ON PLATE COMPILED GROUT HOLE DATA, SOUTH FAULT TEST GROUT PROGRAM

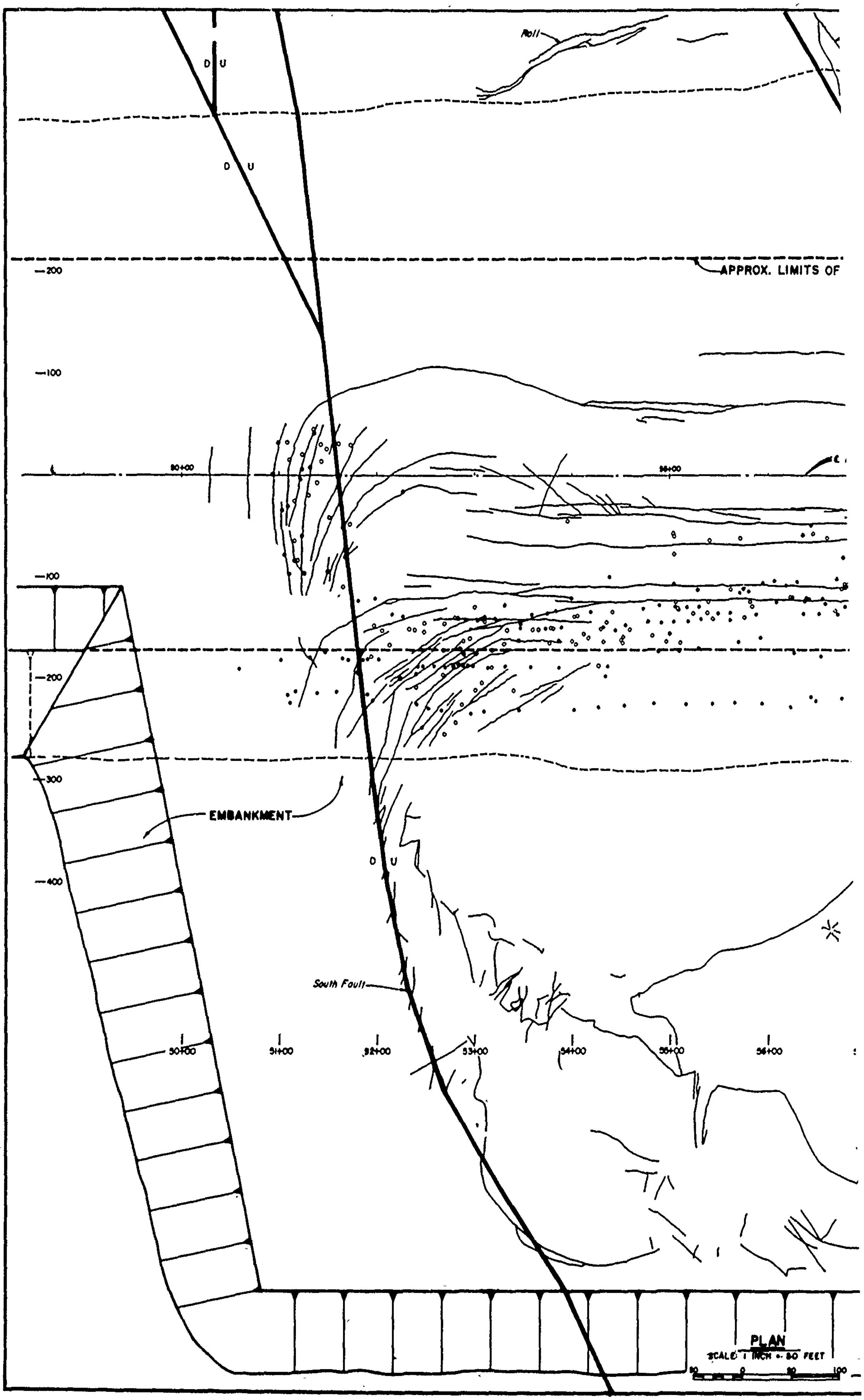
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

GEOLOGIC SECTIONS

UPSTREAM FAULT EXCAVATIONS

SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN. 1963



Roll
D U
D U
APPROX. LIMITS OF
—200
—100
50+00
56+00
—100
—200
—300
—400
EMBANKMENT
D U
South Fault
50+00
51+00
52+00
53+00
54+00
55+00
56+00
PLAN
SCALE: 1 INCH = 80 FEET

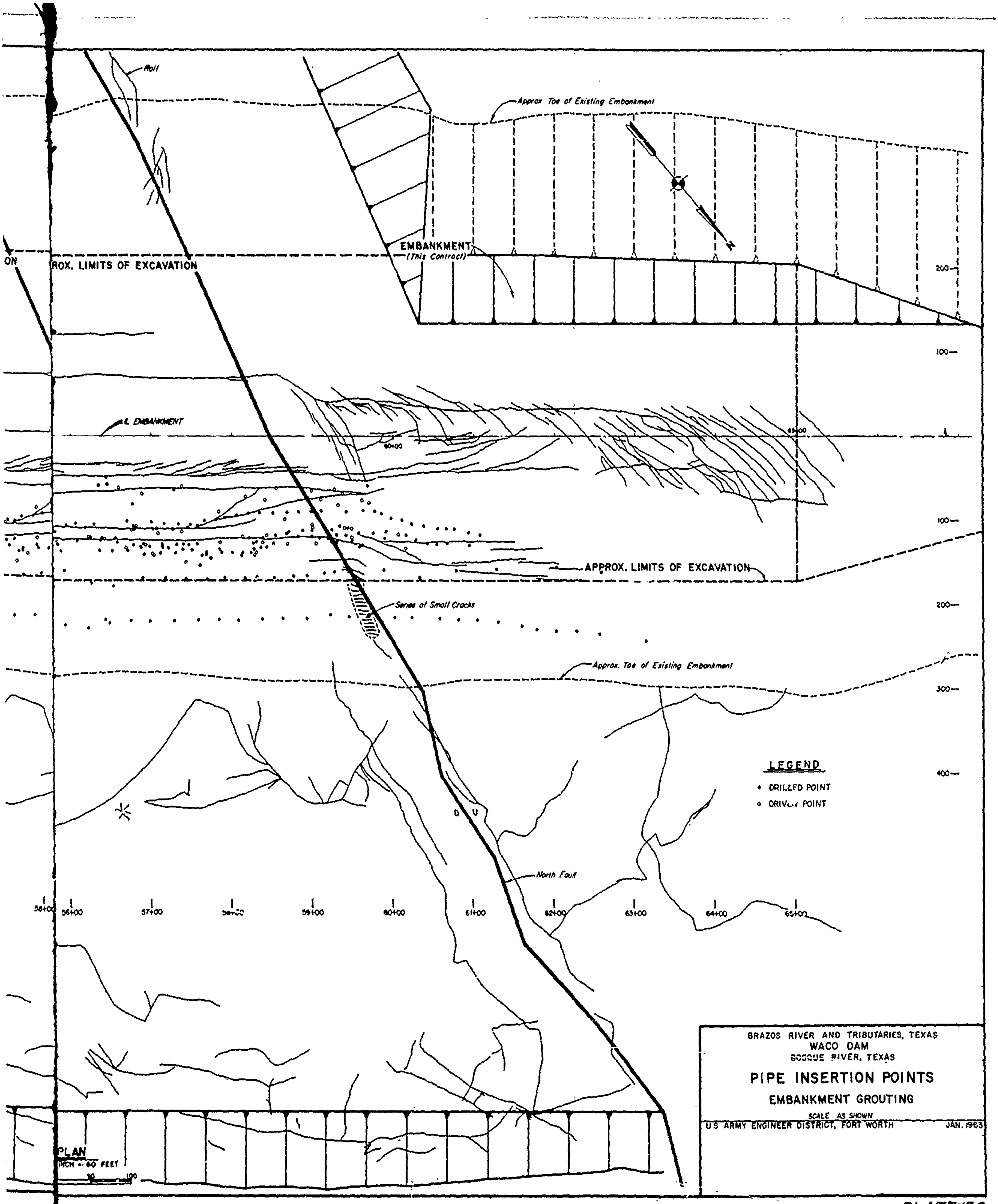
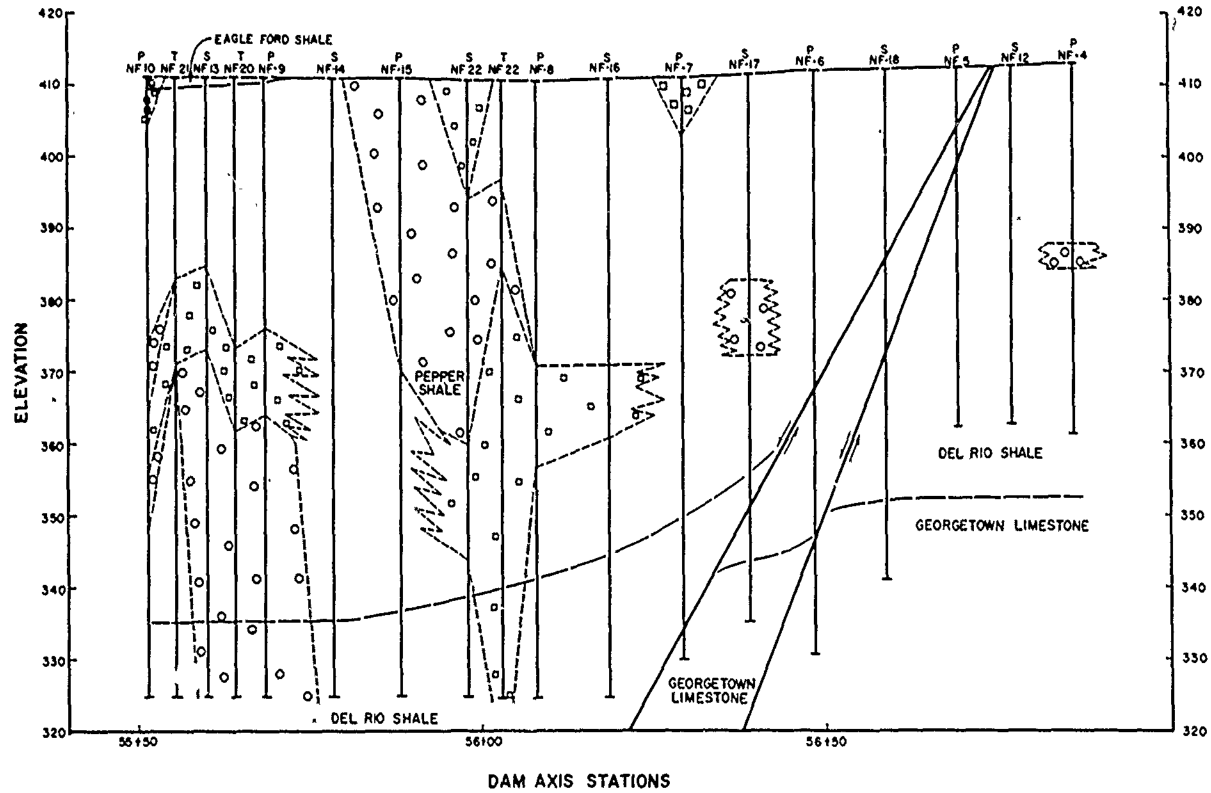
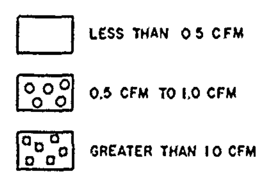

Roll
Approx Toe of Existing Embankment
EMBANKMENT
(This Contract)
ROX. LIMITS OF EXCAVATION
200
100
℄ EMBANKMENT
60+00
65+00
100
APPROX. LIMITS OF EXCAVATION
Series of Small Cracks
200
Approx. Toe of Existing Embankment
300
LEGEND
• DRILLED POINT
○ DRIVEN POINT
400
D U
North Fault
56+00
57+00
58+00
59+00
60+00
61+00
62+00
63+00
64+00
65+00
PLAN
1 INCH = 60 FEET
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS
PIPE INSERTION POINTS
EMBANKMENT GROUTING
SCALE AS SHOWN
US ARMY ENGINEER DISTRICT, FORT WORTH
JAN. 1963

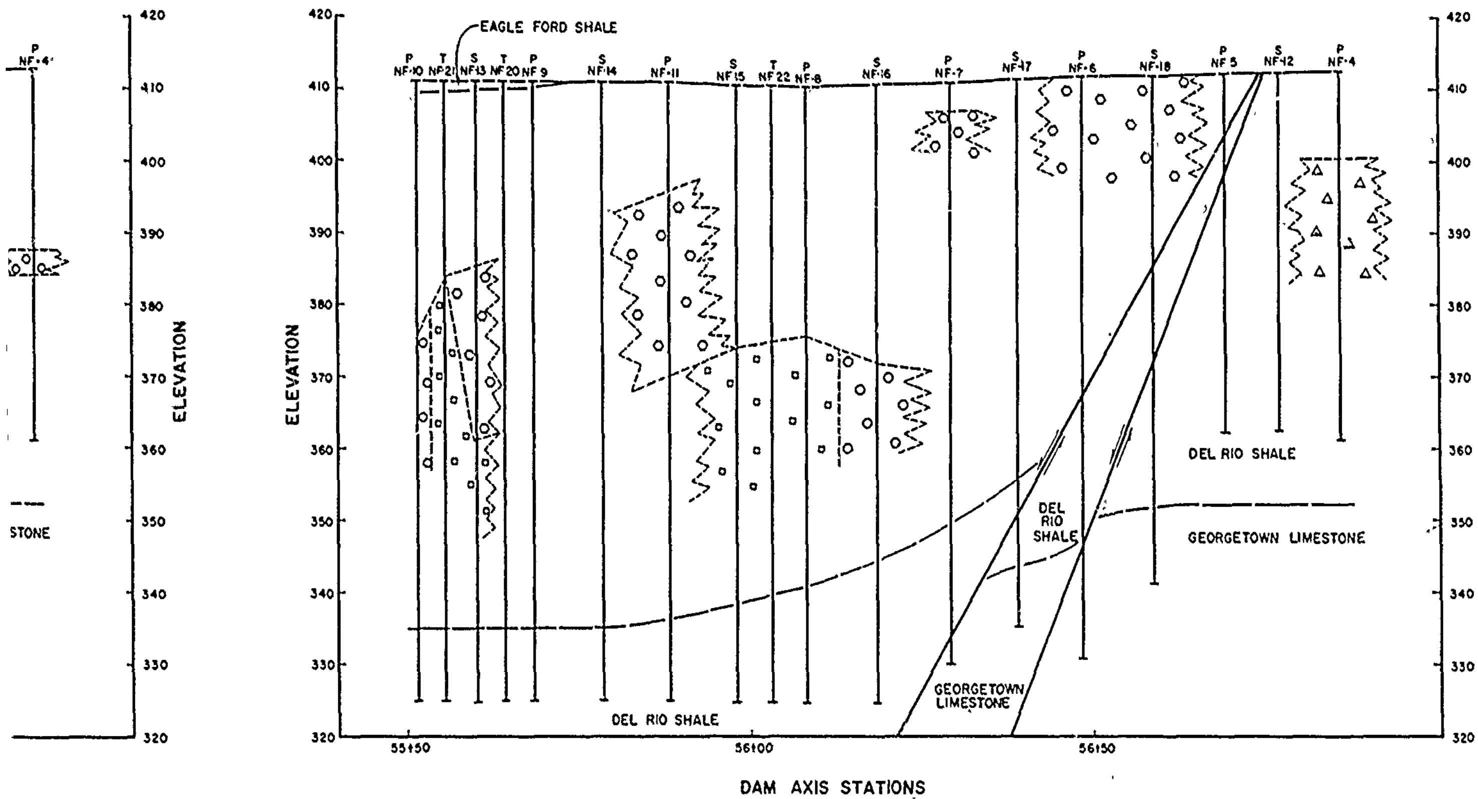
PLATE 56



WATER TEST TAKE DISTRIBUTION

LEGEND-WATER TEST DATA





GROUT TAKE DISTRIBUTION

LEGEND - GROUT TAKE DATA

- 0 SACKS
- 1 TO 25 SACKS
- 25 TO 50 SACKS
- 50 TO 150 SACKS
- GREATER THAN 150 SACKS

BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

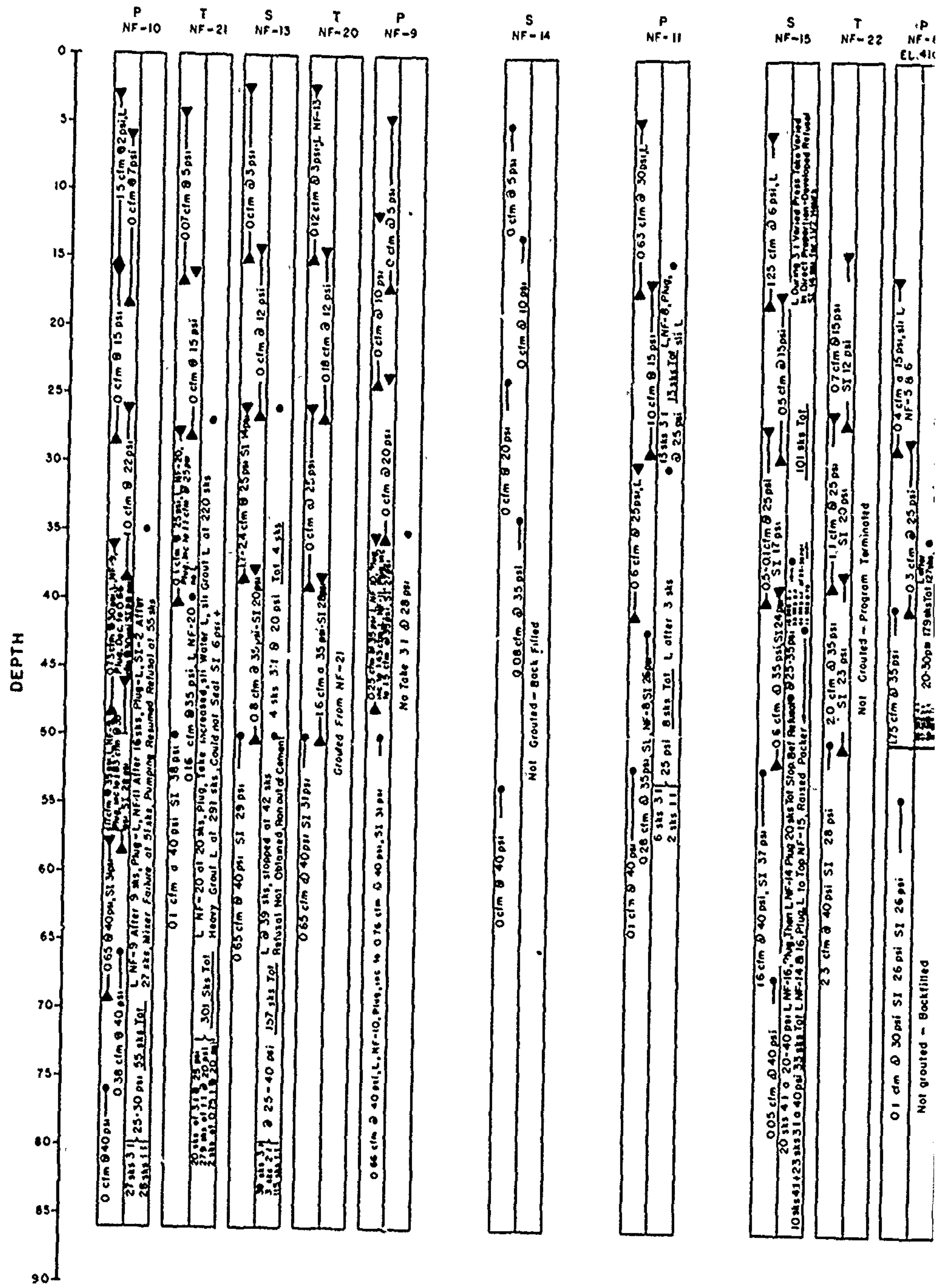
TEST GROUT PROGRAM - NORTH FAULT

DISTRIBUTION OF WATER AND GROUT TAKES

SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH JAN. 1963

CORPS OF ENGINEERS



EXPLANATION OF SYMBOLS

P Denotes primary grout hole

S Denotes secondary grout hole

T Denotes tertiary grout hole

● Location of a packer during grouting or in a single packer water pressure test.

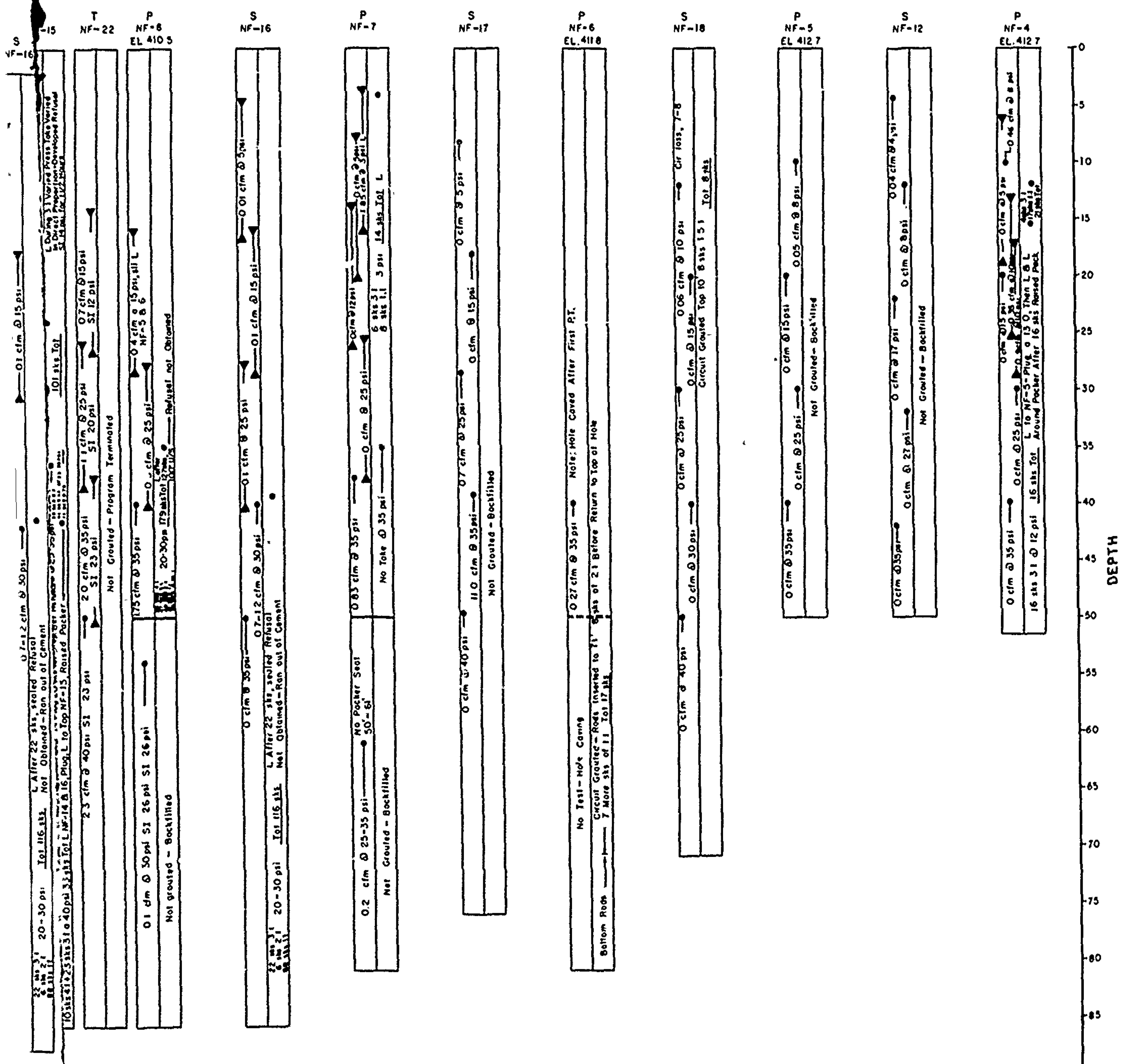
▼▲ Limits of a zone tested during a double packer water pressure test.

L Denotes leakage from the hole during water testing or grouting. The leakage is to the g surface unless a hole number is designatec the leakage is to the designated hole.

3:1 2:1 e°c Denotes the grout mix used in terms of cu feet of water per sack of cement

SI Denotes back pressure in the grout hole wit water test or grout system completely clos

Tot Denotes total grout take during any single



F SYMBOLS

hole
out hole
t hole
during grouting or
ater pressure test.
d during a double
test
the hole during water pressure
e leakage is to the ground
number is designated, where
designated hole.
used in terms of cubic
ck of cement
in the grout hole with the
stem completely closed.
ake during any single grout application.
ion

NOTES

1) The grout holes of the North Fault grout plot were drilled to full depth and then stop tested and grouted. Exceptions were primary holes NF-6, NF-7 and NF-8 which were originally programed for a depth of 50' and subsequently extended.

2) The left column of each boring log presents water pressure test data. The right column presents grout placement data.

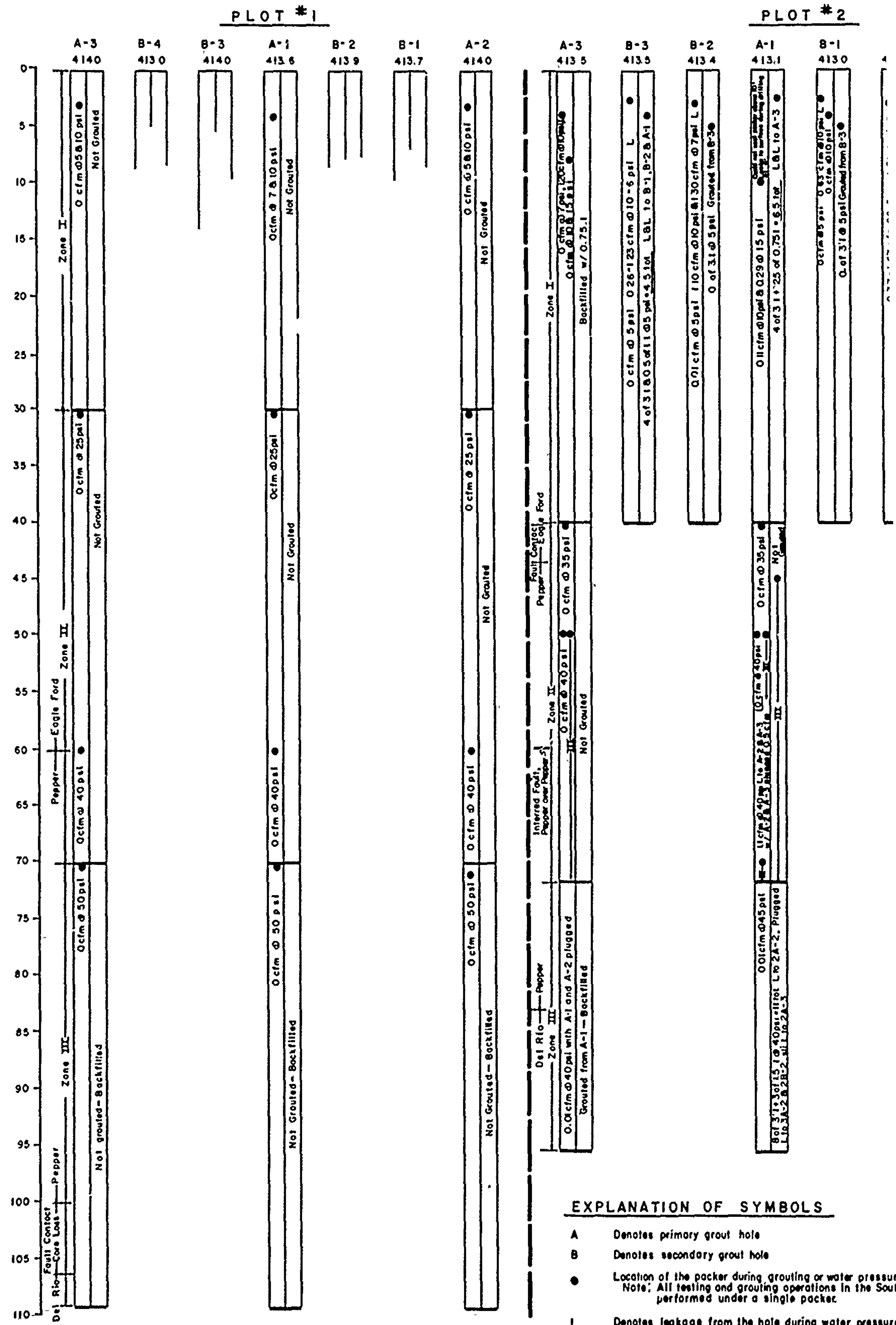
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

TEST GROUT PROGRAM - NORTH FAULT
COMPILED GROUT HOLE DATA

SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH JAN. 1963

CORPS OF ENGINEERS



EXPLANATION OF SYMBOLS

A Denotes primary grout hole

B Denotes secondary grout hole

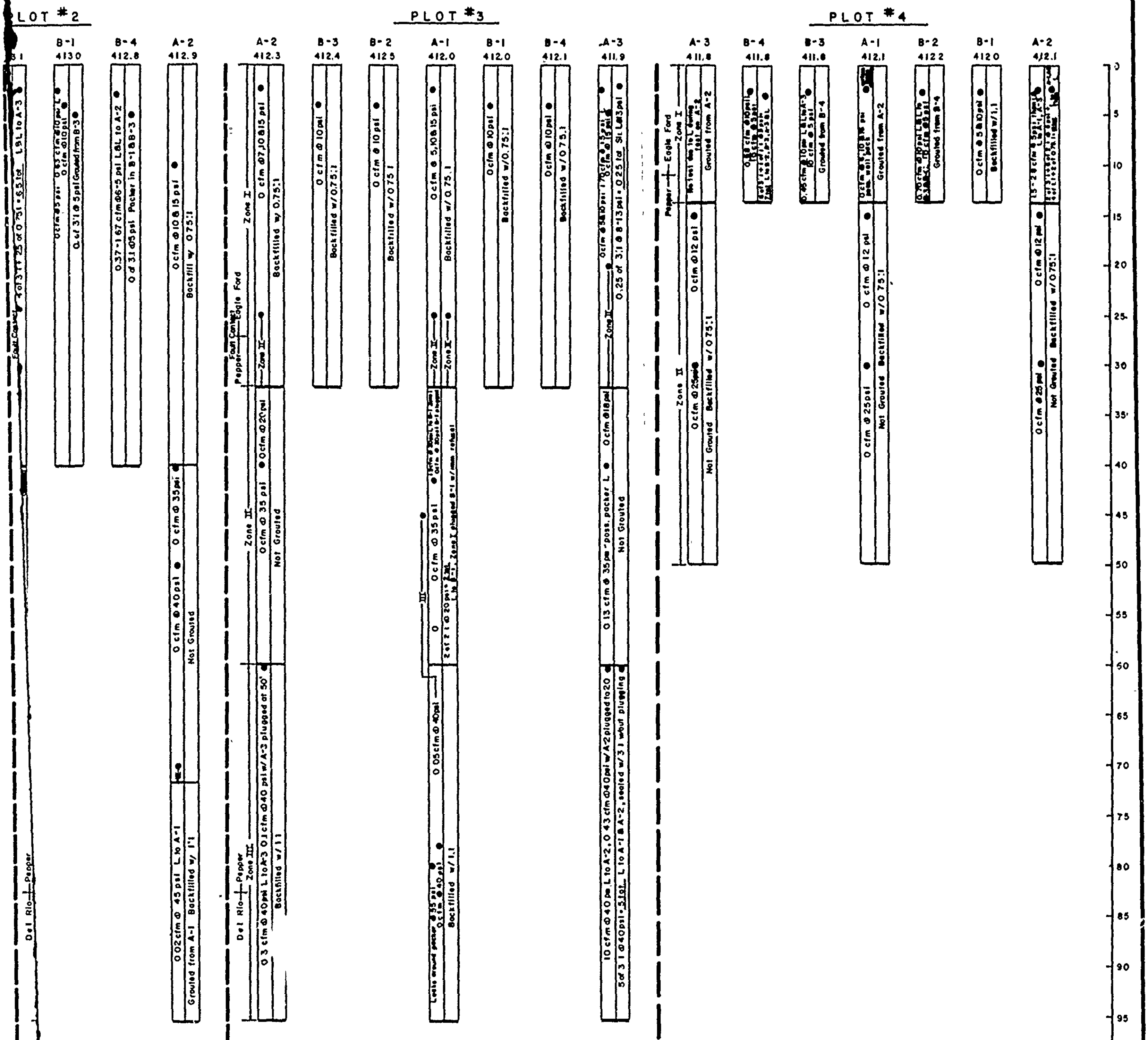
● Location of the packer during grouting or water pressure
Note: All testing and grouting operations in the South
performed under a single packer.

L Denotes leakage from the hole during water pressure
or grouting. The leakage is to the ground surface un
number is designated, where the leakage is to the

3:1 2:1 Denotes the grout mix used in terms of cubic feet of
sack of cement

Tot Denotes total grout take during any single grout op

U.S. ARMY



LS

g or water pressure testing.
rations in the South Fault Program
packer.

ng water pressure testing
ground surface unless a hole
leakage is to the designated hole.

hs of cubic feet of water per

ny single grout application.

NOTES:

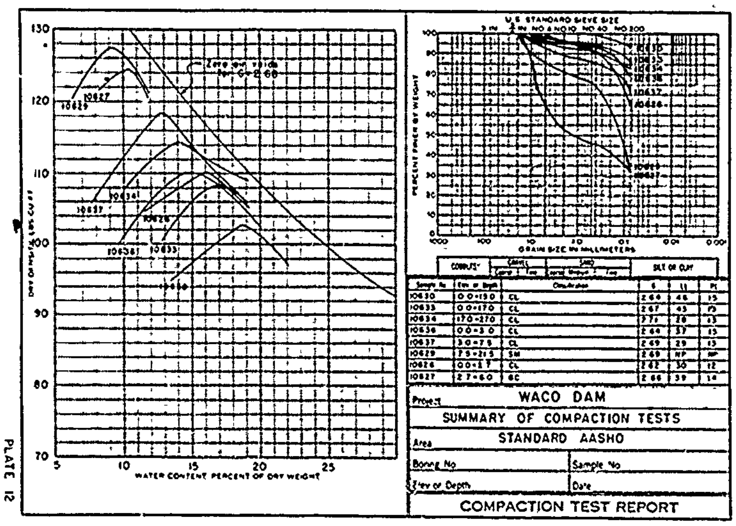
1. The grout holes of the four plots of the South Fault Test Program were drilled and grouted by the stage grouting method, with two zones in Plot #4 and three zones in Plots #1, 2 & 3. The limits of the zones are shown by double horizontal lines in the boring column.
2. The first primary hole (A-1) of each plot was cored. Geologic contacts are shown at the left of each plot.
3. The left column of each boring log presents water pressure test data. The right column presents grout placement data.

BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

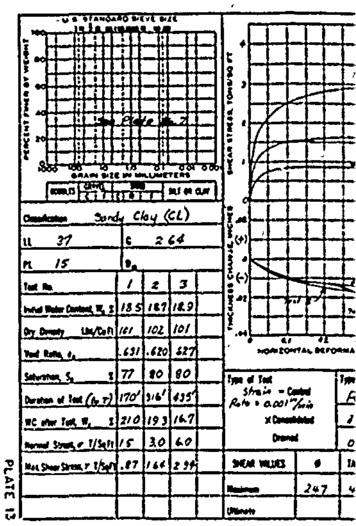
TEST GROUT PROGRAM-SOUTH FAULT
COMPILED GROUT HOLE DATA

SCALE AS SHOWN

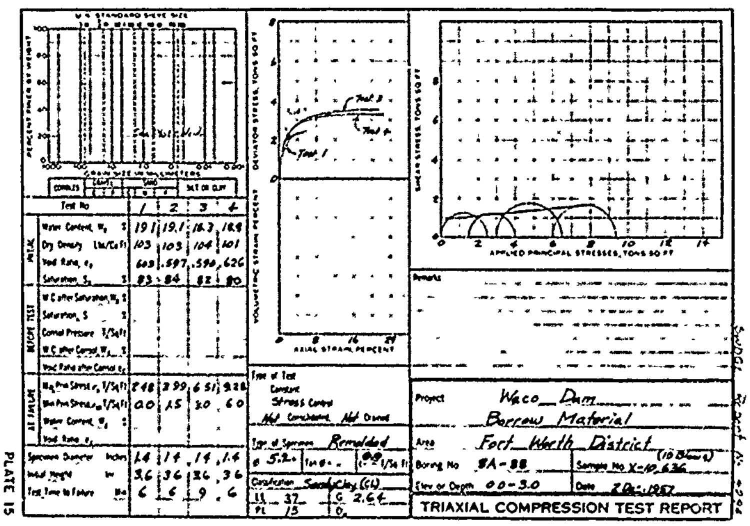
U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN. 1963



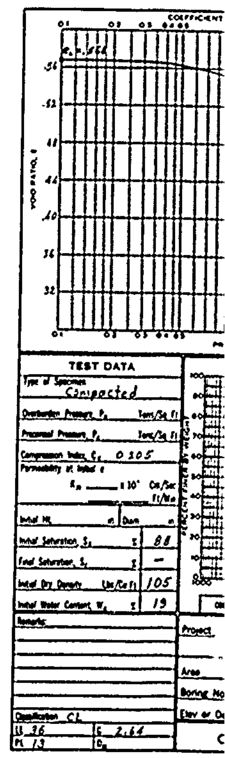
WACO DAM
SUMMARY OF COMPACTION TESTS
STANDARD AASHO
COMPACTION TEST REPORT
WATER CONTENT PERCENT OF DRY WEIGHT
U.S. STANDARD SIEVE SIZE
GRAIN SIZE IN MILLIMETERS
Zero air voids for G=2.60
PLATE 12



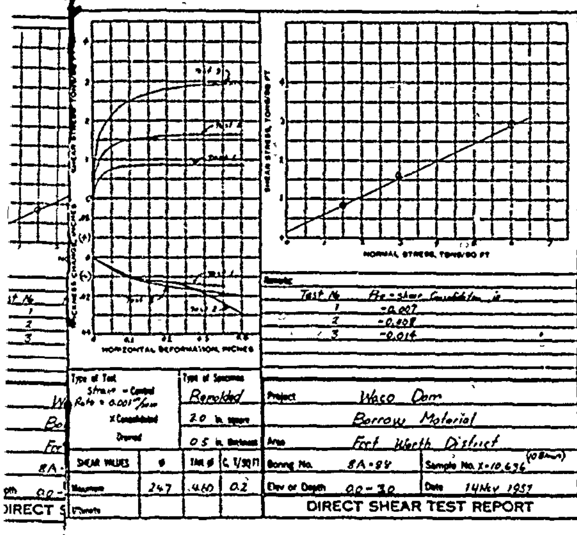
Sandy Clay (CL)
PLATE 13

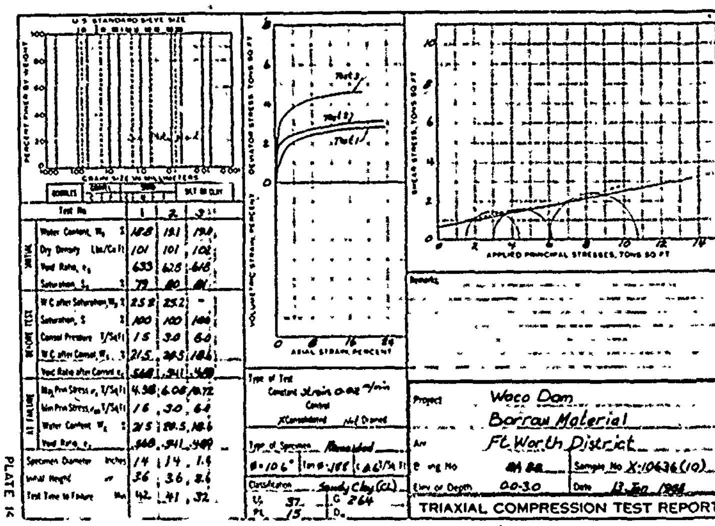


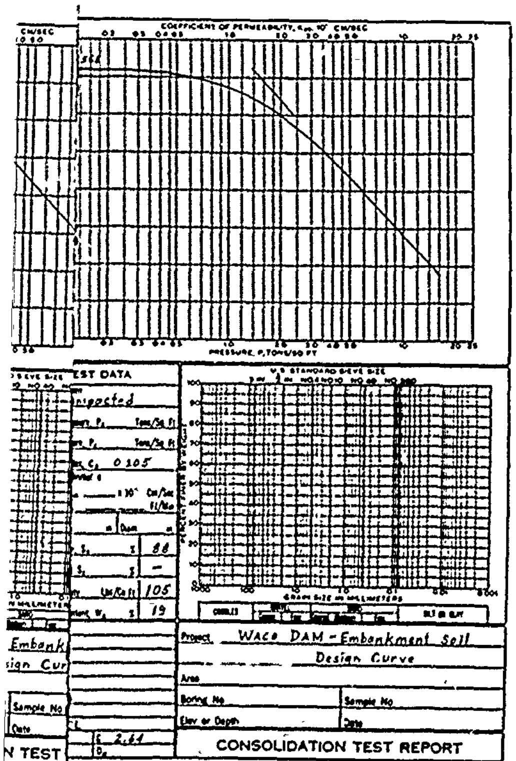
Waco Dam
Borrow Material
Fort Worth District
TRIAXIAL COMPRESSION TEST REPORT
APPLIED PRINCIPAL STRESSES, TONS SQ FT
AXIAL STRAIN PERCENT
Remolded
Sandy Clay (CL)
PLATE 15



TEST DATA
Compacted







SYMBOL	NO.	ACTION	DATE	DESCRIPTION OF REVISION

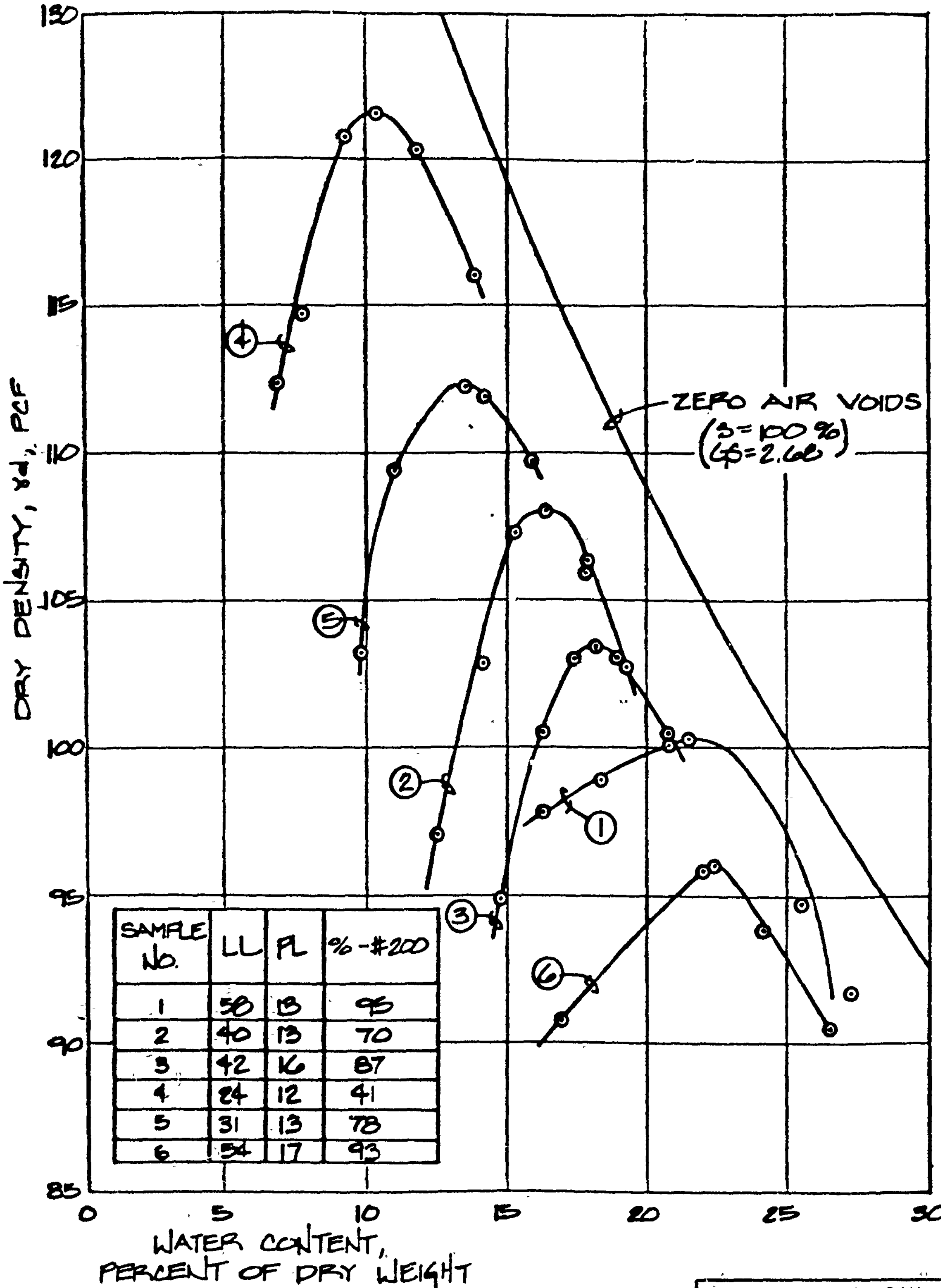
U.S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

DESIGNED BY:
DRAWN BY:
REVIEWED BY:
SUBMITTED BY:
ENGINEER:

WACO DAM
BOSQUE RIVER, TEXAS

TYPICAL TEST RESULTS
ORIGINAL DESIGN

SOL. NO. | DATED:
CONTR. NO
DRAWING NUMBER | SHEET NO. OF | SEQUENCE NO.

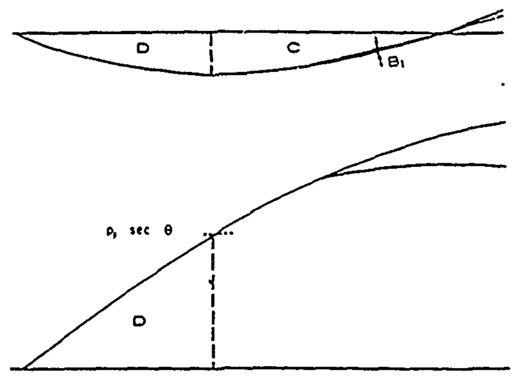


SAMPLE No.	LL	PL	%-#200
1	58	13	95
2	40	13	70
3	42	16	87
4	24	12	41
5	31	13	78
6	54	17	93

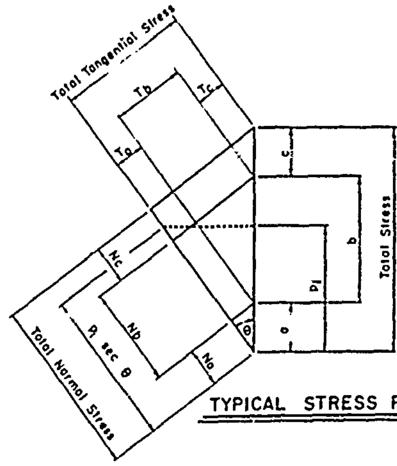
WACO DAM
BOSQUE RIVER, TEXAS

COMPACTION TEST RESULTS
(CE 12)
FROM SPILLWAY EXCAVATION

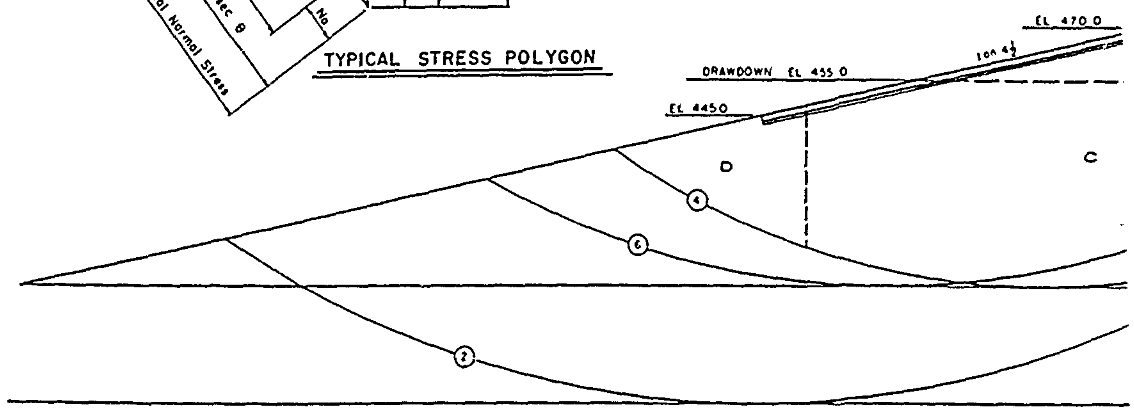
a = Stress due to portion of embankment below drawdown EL 455 0
b = Stress due to portion of embankment above drawdown and below phreatic line
c = Stress due to portion of embankment above phreatic line
T = Tangential Stress - subscript corresponds to respective total stress
N = Normal Stress - subscript corresponds to respective total stress
Vector diagrams were measured in feet of soil



NORMAL



TYPICAL STRESS POLYGON



DESIGN DATA-EMBANKMENT MATERIAL

γ_{dry} = 105 pcf (95 % std. AASHO)
γ_{moist} = 125 pcf w = 19 %
γ_{sat} = 128 pcf w = 22 %
γ_{buoyed} = 66 pcf

SHEAR STRENGTHS	φ°	C (tsf)
Consolidated Drained	25°	0.1
Consolidated Undrained	12°	0 4
Unconsolidated Undrained	3°	1.0

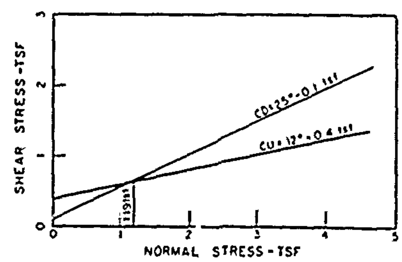
P_1(CD-CU) for rapid drawdown = 1 19 tsf = 36' of buoyed soil

DESIGN DATA-FOUNDATION MATERIAL (RIGHT FLOODPLAIN)

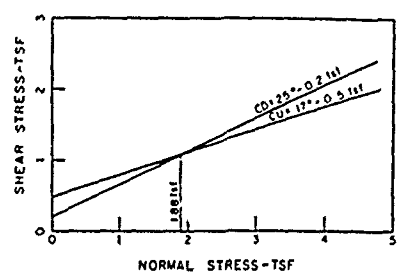
γ_{dry} 100 pcf
γ_{moist} 122 pcf w = 22 %
$\gamma_{sat.}$ 125 pcf w = 25 %
γ_{buoyed} 63 pcf

SHEAR STRENGTHS:	φ°	C (tsf)
Consolidated Drained	25°	0 2
Consolidated Undrained	17°	0 5
Unconsolidated Undrained	5°	1.5

P_1(CD-CU) for rapid drawdown = 1 88 tsf = 60' of buoyed soil



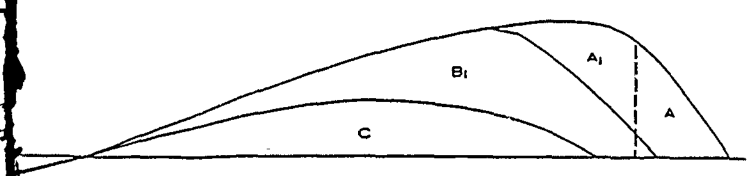
EMBANKMENT MATERIAL



FOUNDATION MATERIAL

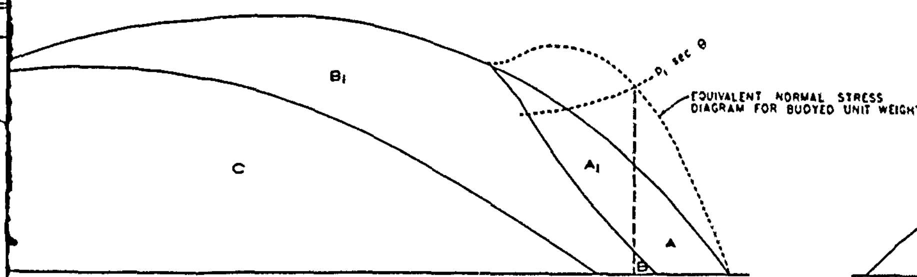
COMBINED STRENGTH ENVELOPES
FOR RAPID DRAWDOWN CONDITION

STABILI

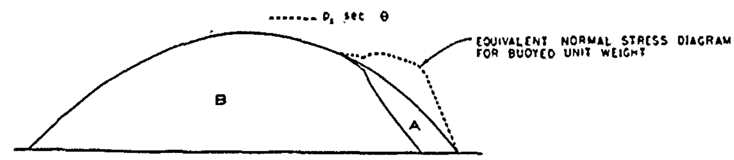


TANGENTIAL STRESSES - CIRCLE 4

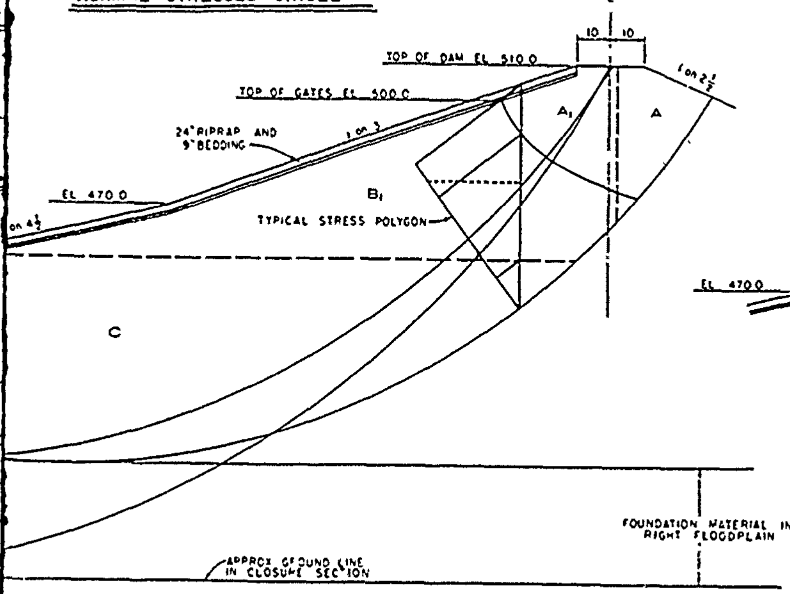
TANGENTIAL STRESSES - CIRCLE 8

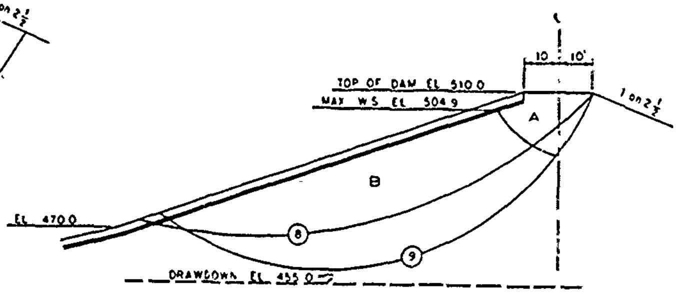


NORMAL STRESSES - CIRCLE 4



NORMAL STRESSES - CIRCLE 8





STABILITY ANALYSIS - CIRCLE 4 - RAPID DRAWDOWN CONDITION (CD-CU STRENGTH)

SEGMENT	AREA-SQ FT	UNIT WEIGHT-PCF	FORCE-LBS	ARC LENGTH-FT
Normal Forces				
A	404	125	50,500	37
B	24	66	1,584	8
D	1068	66	70,488	64
			122,572	109
A_1	656	125	82,000	
B_1	4096	66	270,336	17
C	9312	66	614,592	236
			966,928	253
Tangential Forces				
A	464	125	58,000	
B	16	128	2,048	
D	-480	66	-31,680	
A_1	596	125	74,500	
B_1	1784	128	228,352	
C	976	66	64,416	
			395,636	

$$SF = \frac{122.6 \tan 25^\circ + 109(0.2) + 966.9 \tan 12^\circ + 253(0.8)}{395.6}$$

$$= \frac{122.6(0.466) + 109(0.2) + 966.9(0.213) + 253(0.8)}{395.6}$$

$$= \frac{57.1 + 21.8 + 205.9 + 202.4}{395.6}$$

$$= \frac{487.2}{395.6}$$

$$= 1.23$$

Rapid Drawdown - Circle 6 - SF = 1.28
(CD-CU Strength) Circle 2 - SF = 1.35 (Closure section)
Circle 2 - SF = 1.73 (Right floodplain)
Post Construction - Circle 2 - SF = 1.80 (Closure section)
(UU Strength) SF = 2.47 (Right floodplain)
- Circle 4 - SF = 1.97
- Circle 6 - SF = 2.16

STABILITY ANALYSIS - CIRCLE 8 - RAPID DRAWDOWN CONDITION (CD-CU STRENGTH)

SEGMENT	AREA-SQ. FT.	UNIT WEIGHT-PCF	FORCE-LBS	ARC LENGTH-FT
Normal Forces				
A	210	125	26,250	22
B	2694	66	177,804	129
			204,054	151
Tangential Forces				
A	236	125	29,500	
B(+)	675	128	86,400	
B(-)	149	128	-19,072	
			96,828	

$$SF = \frac{204.1 \tan 25^\circ + 151.0(0.2)}{96.8}$$

$$= \frac{204.1(0.466) + 151.0(0.2)}{96.8}$$

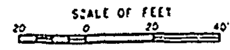
$$= \frac{95.1 + 30.2}{96.8}$$

$$= \frac{125.3}{96.8}$$

SF = 1.29

Rapid Drawdown - Circle 9 - SF = 1.31
(CD-CU Strength)
Post Construction - Circle 9 - SF = 4.32
(UU Strength) - Circle 8 - SF = 3.48

SCALE OF FEET

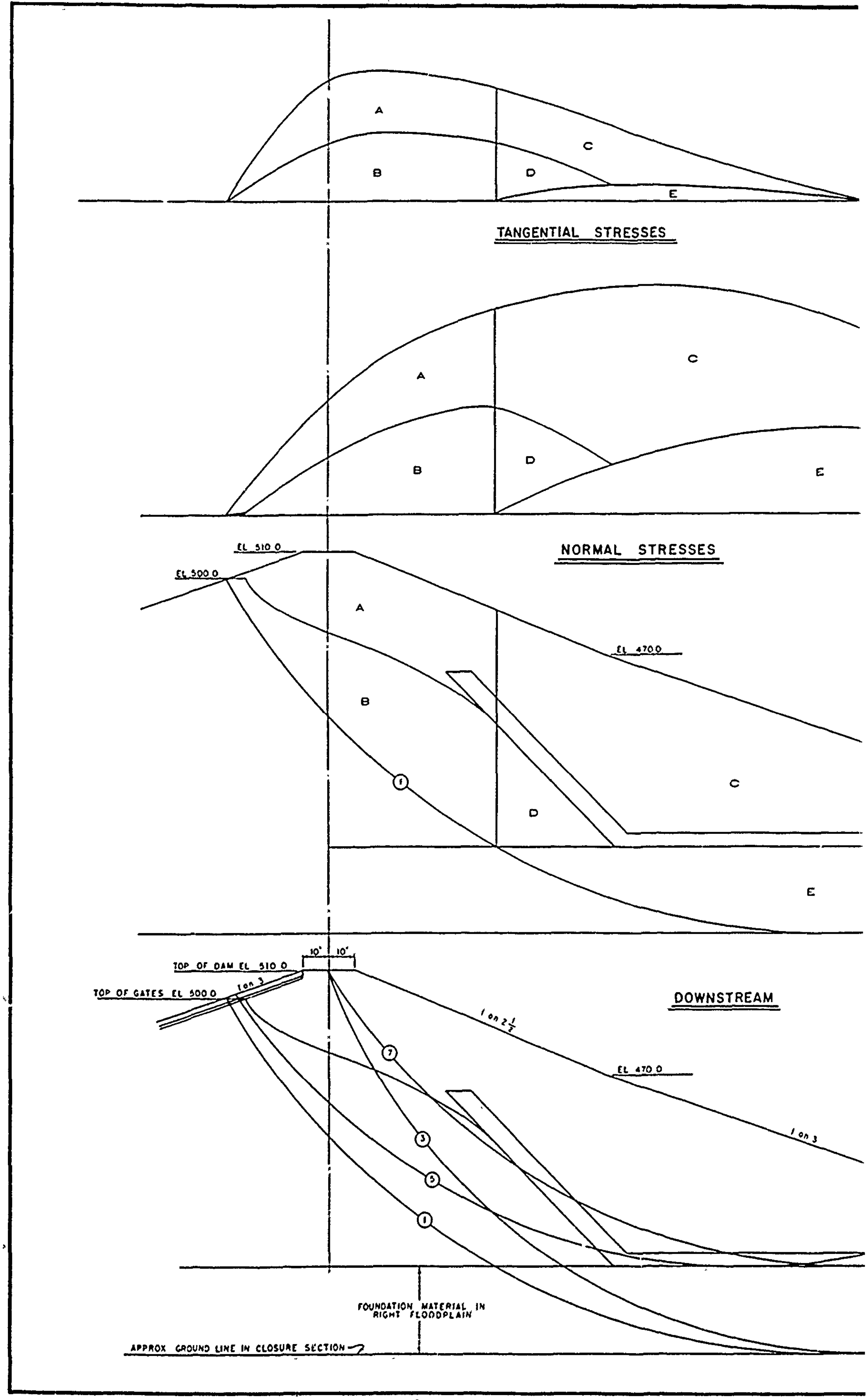


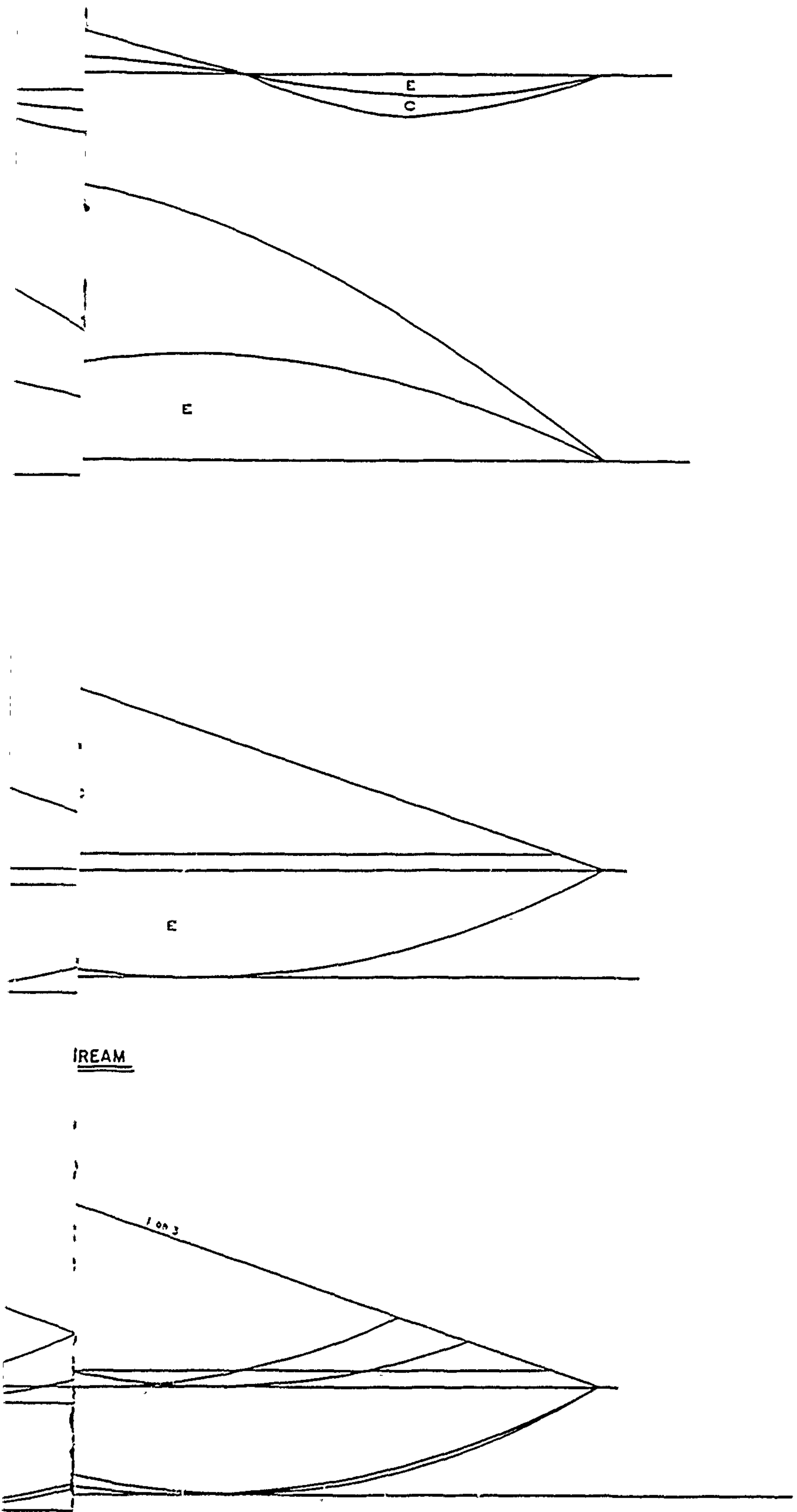
BRAZOS RIVER BASIN, TEXAS
WACO RESERVOIR
BOSQUE RIVER, TEXAS

STABILITY ANALYSIS
UPSTREAM SLOPES
ORIGINAL DESIGN

FORT WORTH DISTRICT FORT WORTH, TEXAS JUNE 1958

CORPS OF ENGINEERS





STABILITY ANALYSIS - CIRCLE 1 - STEADY STATE SEEPAGE (CD STRENGTH)

SEGMENT	AREA-SQ FT	UNIT WEIGHT-PCF	FORCE-LBS	ARC LENGTH-FT
Normal Forces				
AC	12,336	125	1,542,000	
BDE	9,300	66	613,800	422
			2,155,800	422
Tangential Forces				
AC	3,444	125	430,500	
BDE	2,628	128	336,384	
			766,884	

$$SF = \frac{2155.8 \tan 25^\circ + 422.0(0.2)}{766.9} = \frac{1089.0}{766.9}$$

SF = 1.42 Closure Section

Steady State Seepage - Circle 1 - SF = 1.48 Floodplain Section
(CD Strength) - Circle 5 - SF = 1.75
- Circle 7 - SF = 1.73
- Circle 3 - SF = 1.50 Floodplain Section
- SF = 1.44 Closure Section

STABILITY ANALYSIS - CIRCLE 1 - POST CONSTRUCTION (UU STRENGTH)

SEGMENT	AREA-SQ.FT	UNIT WEIGHT-PCF	FORCE-LBS	ARC LENGTH-FT
Normal Forces				
Σ	21636	125	2,704,500	422
Tangential Forces				
Σ	6176	125	772,000	

$$SF = \frac{2704.5 \tan 3^\circ + 422.0(2.0)}{772.0}$$

$$= \frac{2704.5(0.052) + 422.0(2.0)}{772.0}$$

$$= \frac{1406 + 844}{772.0} = \frac{984.6}{772.0}$$

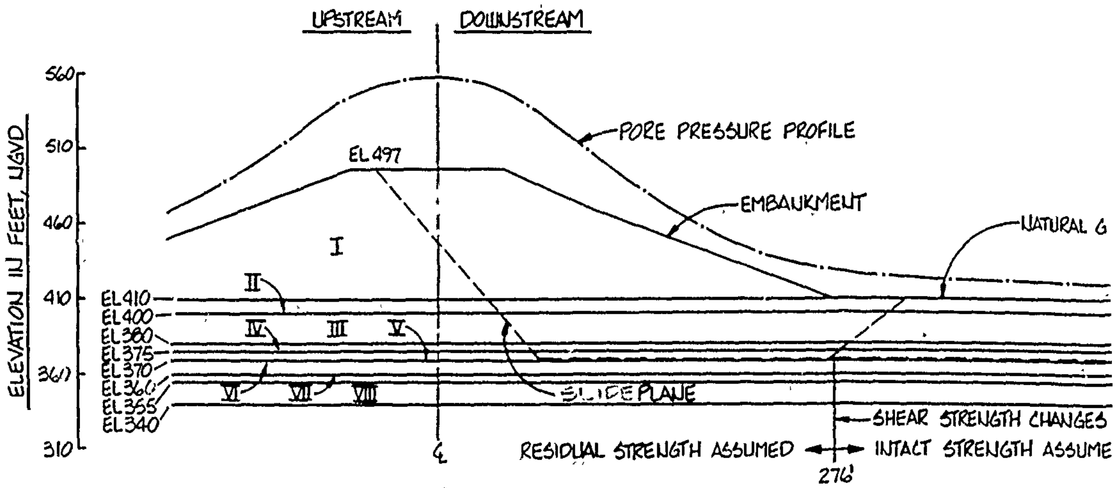
SF = 1.28 Closure Section

Post Construction - Circle 1 - SF = 1.72 Floodplain Section
(UU Strength) - Circle 3 - SF = 1.90 Floodplain Section
- SF = 1.42 Closure Section
- Circle 5 - SF = 1.93
- Circle 7 - SF = 1.98

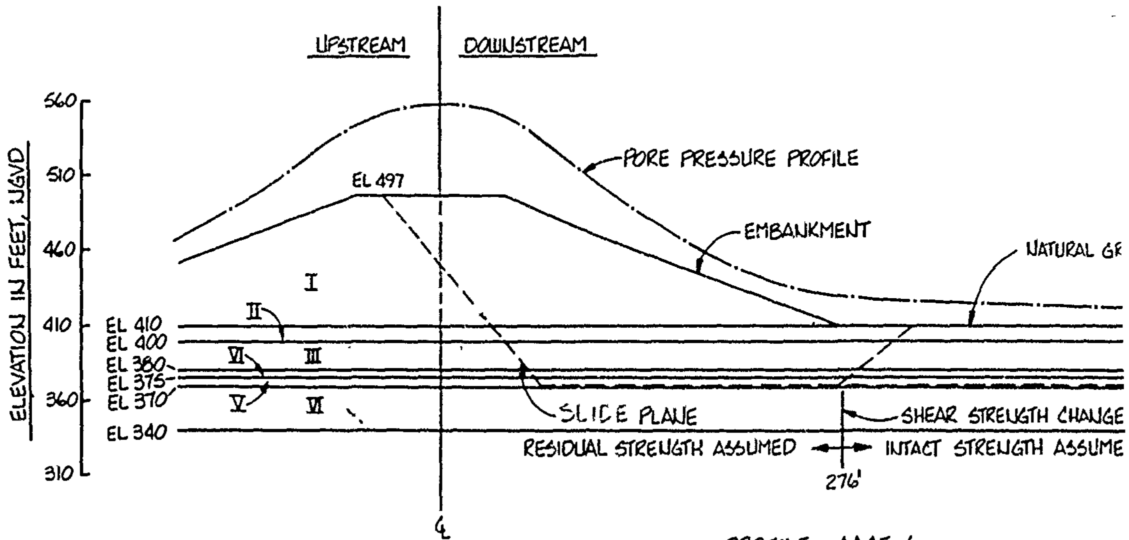
BRAZOS RIVER BASIN, TEXAS
WACO RESERVOIR
BOSQUE RIVER, TEXAS

STABILITY ANALYSIS
DOWNSTREAM SLOPES
ORIGINAL DESIGN

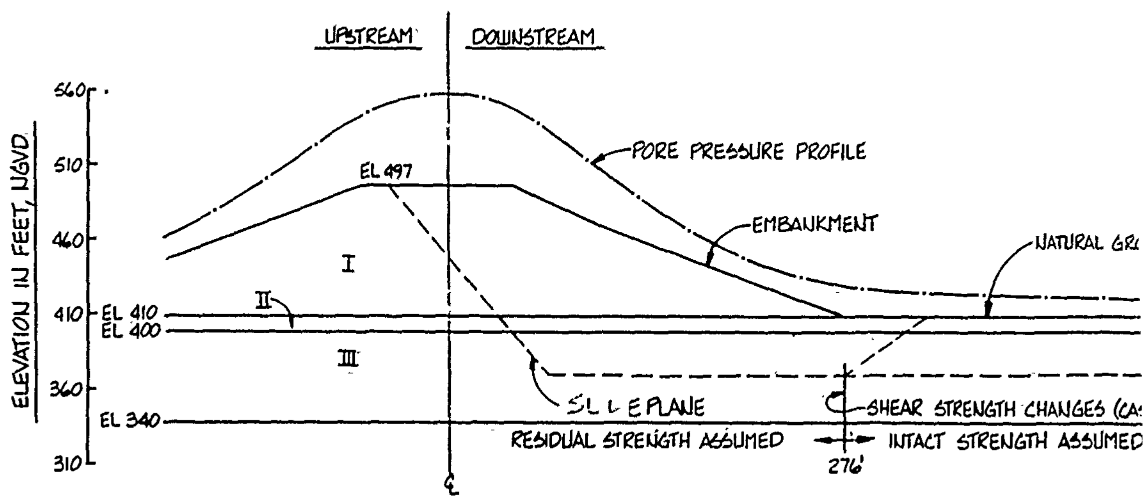
FORT WORTH DISTRICT FORT WORTH, TEXAS JUNE 1958



PROFILE - CASES 1 THRU 5

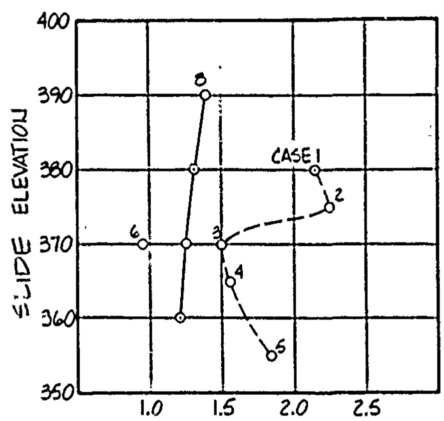


PROFILE - CASE 6

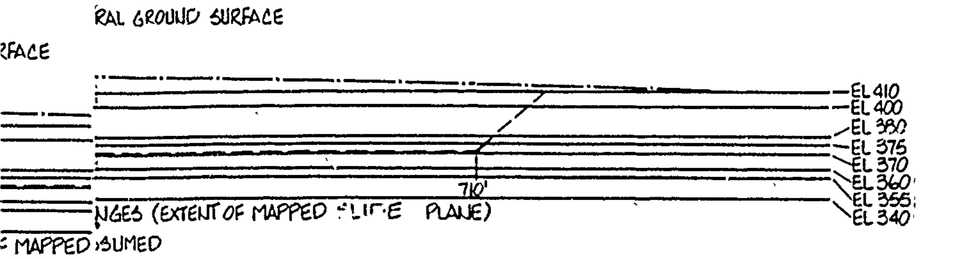


PROFILE - CASES 7 AND 8

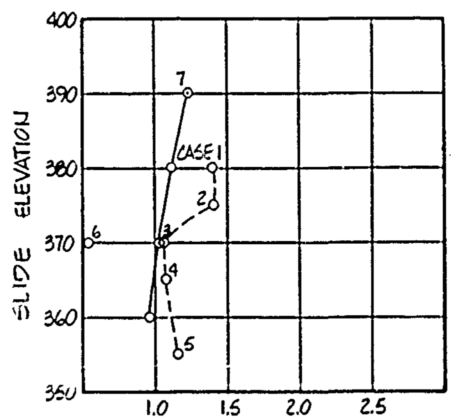
LONG SLIDE PATH



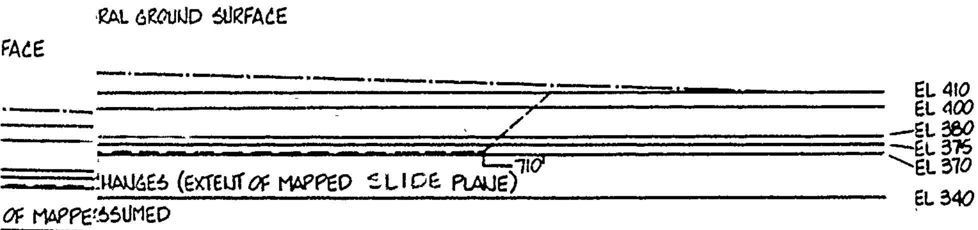
SAFETY FACTOR
(1961 PORE PRESSURE)



SHORT SLIDE PATH

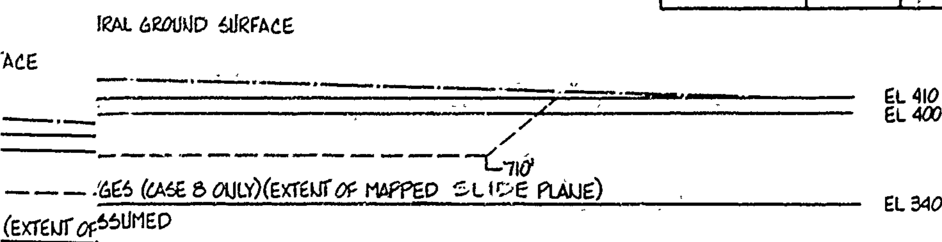


SAFETY FACTOR
(1961 PORE PRESSURE)



SHEAR STRENGTH SUMMARY

STRATUM	CASE 1 Φ° C, PSF	CASE 2 Φ° C, PSF	CASE 3 Φ° C, PSF	CASE 4 Φ° C, PSF	CASE 5 Φ° C, PSF	CASE 6 Φ° C, PSF	CASE 7 Φ° C, PSF	CASE 8 Φ° C, PSF
I EMBANKMENT	3 2000	3 2000	3 2000	3 2000	3 2000	3 1000	3 2000	3 2000
II 400-410	20 400	20 400	20 400	20 400	20 400	20 400	20 400	20 400
III 380-400	18 580	18 580	18 580	18 580	18 580	8 0* 14 400	16 400	8 0* 16 400
IV 375-380	23 760	23 760	23 760	23 760	23 760	8 0* 14 400	16 400	8 0* 16 400
V 370-375		23 1000	18 600	18 600	18 600	8 0* 14 400	16 400	8 0* 16 400
VI 360-370			14 400	15 480	15 500	14 400	16 400	8 0* 16 400
VII 355-360				15 500	21 620	14 400	16 400	8 0* 16 400
VIII 340-355					22 640		16 400	8 0* 16 400



50 0 50 100
SCALE IN FEET

* THE STRENGTH CHANGES AT 276 FEET DOWNSTREAM. THE STRENGTH CHANGES FROM LOW TO HIGH SIGNIFYING THE CHANGE FROM RESIDUAL TO INTACT STRENGTH.

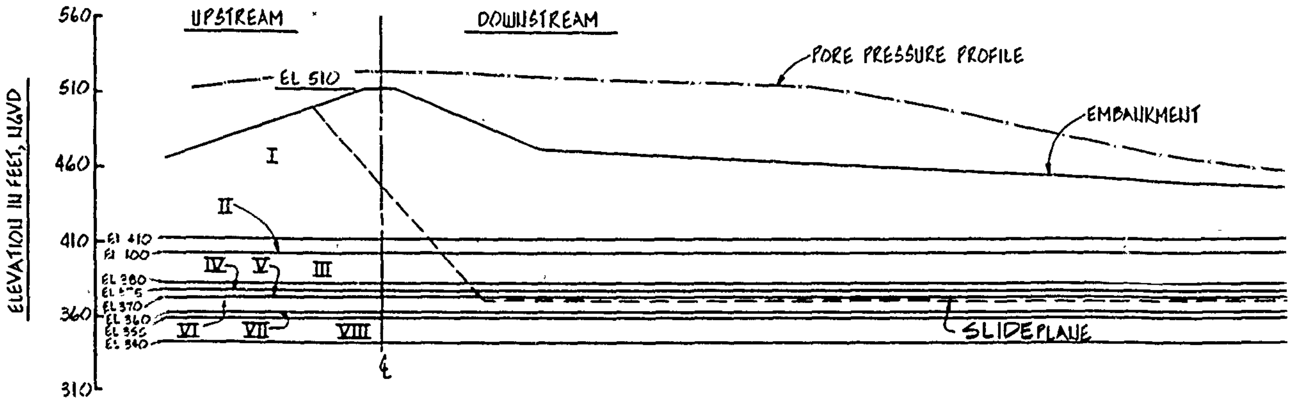
U.S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

DESIGNED BY:
DRAWN BY:
REVIEWED BY:
SUBMITTED BY:

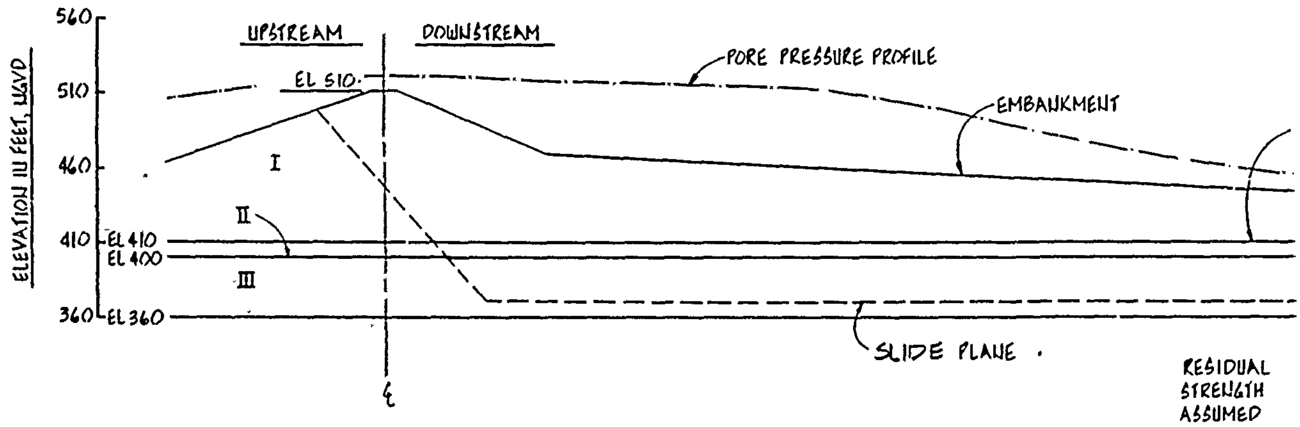
WACO DAM
BOSQUE RIVER, TEXAS

STABILITY ANALYSIS
1961 CONDITION
(PRE - SLIDE)

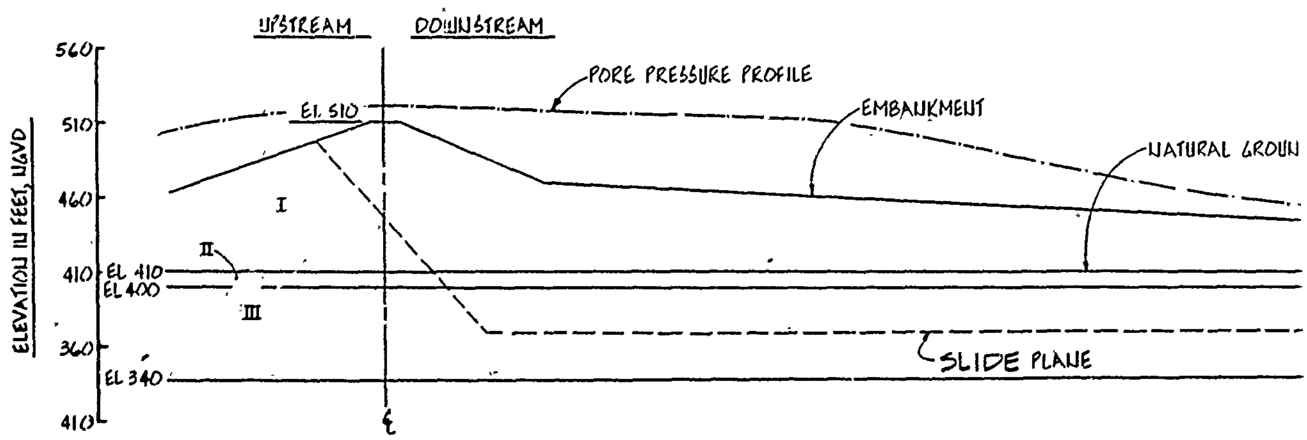
INV. NO.
DATED:
CONTR. NO.
DRAWING NUMBER
SHEET NO.



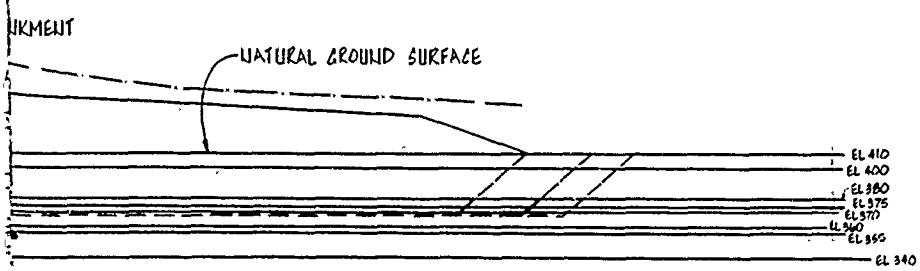
PROFILE CASE 1 AND 5

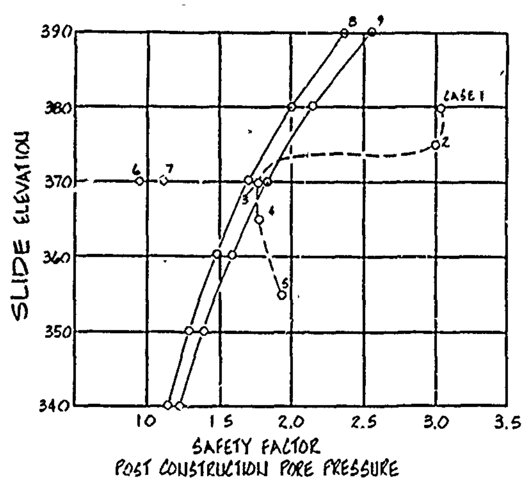


PROFILE CASE 6 AND 7



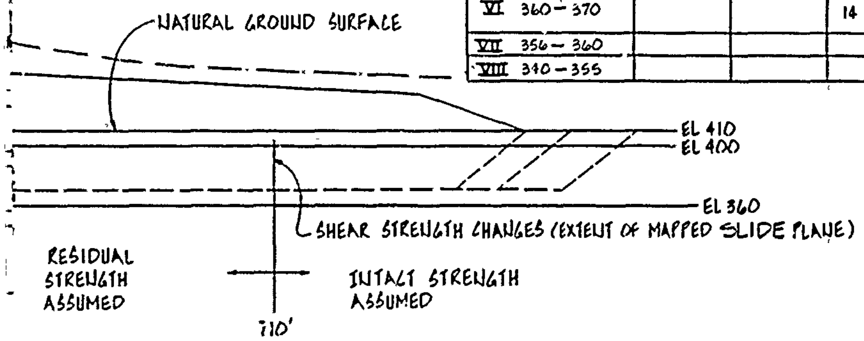
PROFILE CASE 8 AND 9





SHEAR STRENGTH SUMMARY

STRATUM	CASE 1 φ° C,PSF	CASE 2 φ° C,PSF	CASE 3 φ° C,PSF	CASE 4 φ° C,PSF	CASE 5 φ° C,PSF	CASE 6 φ° C,PSF	CASE 7 φ° C,PSF	CASE 8 φ° C,PSF	CASE 9 φ° C,PSF
I EMBANKMENT	3 2000	3 2000	3 2000	3 2000	3 2000	3 1000	3 2000	3 2000	3 2000
II 400 – 410	20 400	20 400	20 400	20 400	20 400	20 400	20 400	20 400	20 400
III 380 – 410	18 580	18 580	18 580	18 580	18 580	8 0* 14 400	8 0* 14 400	14 400	16 400
IV 375 – 380	23 760	23 760	23 760	23 760	23 760	8 0* 14 400	8 0* 14 400	14 400	16 400
V 370 – 375		23 1000	18 600	18 600	18 600	8 0* 14 400	8 0* 14 400	14 400	16 400
VI 360 – 370			14 400	15 480	15 500	8 0* 14 400	8 0* 14 400	14 400	16 400
VII 356 – 360				15 500	21 620			14 400	16 400
VIII 340 – 355					22 640			14 400	16 400



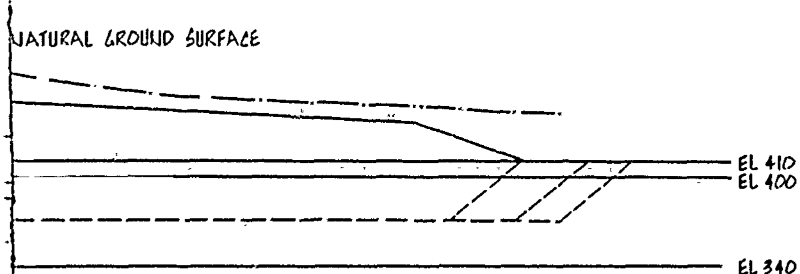
GENERAL NOTES:

1 MINIMUM COMPUTED FACTORS OF SAFETY PRESENTED ARE FOR THE CRITICAL COMBINATION OF ACTIVE AND PASSIVE WEDGE LOCATIONS.

2. FACTORS OF SAFETY WERE COMPUTED USING WESLIB PROGRAM 55W039 (41-Z5-039), WEDGE METHOD WITH EXCESS PORE PRESSURE. THE STABILITY ANALYSES WERE CONDUCTED USING THE HONEYWELL 6000 TIMESHARING COMPUTER AT DALLAS, TEXAS THE PROGRAM PERFORMS WEDGE METHOD STABILITY ANALYSES IN ACCORDANCE WITH EM 1110-2-1902 ("STABILITY OF EARTH AND ROCKFILL DAMS" DATED 1 APRIL 1970) EXCEPT THE PROGRAM USES HORIZONTAL EARTH FORCE DIRECTION THROUGHOUT THE ACTIVE WEDGE. THIS VERY SLIGHT DEVIATION IS CONSERVATIVE BUT HAS MINIMAL EFFECT ON COMPUTED FACTORS OF SAFETY IN COMPARISON TO ANY NUMBER OF OTHER ASSUMPTIONS COMMONLY MADE IN STABILITY ANALYSES.

3 THE EMBANKMENT WAS ASSUMED TO BE HOMOGENEOUS AND NOT A ZONED EMBANKMENT. S STRENGTHS WERE USED FOR ALL MATERIALS IN THIS ANALYSIS. THE PORE WATER PRESSURE PROFILE FOR EACH SET OF CASES IS SHOWN WITH EACH CROSS-SECTION.

*1 THE STRENGTH CHANGES 710 FEET DOWNSTREAM, THE STRENGTH CHANGES FROM LOW TO HIGH SIGNIFYING THE CHANGE FROM RESIDUAL TO INTACT STRENGTH.



U.S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

DESIGNED BY:
DRAWN BY:
REVIEWED BY:
SUBMITTED BY:
ENGINEER:

WACO DAM
BOSQUE RIVER, TEXAS
STABILITY ANALYSIS
1964 CONDITION
END OF CONSTRUCTION

INV. NO.
DATED:
CONTR. NO.
DRAWING NUMBER
SHEET NO.

DESIGN DATA

UNIT WEIGHT SOIL = 130 pcf
UNIT WEIGHT WATER = 62.5 pcf

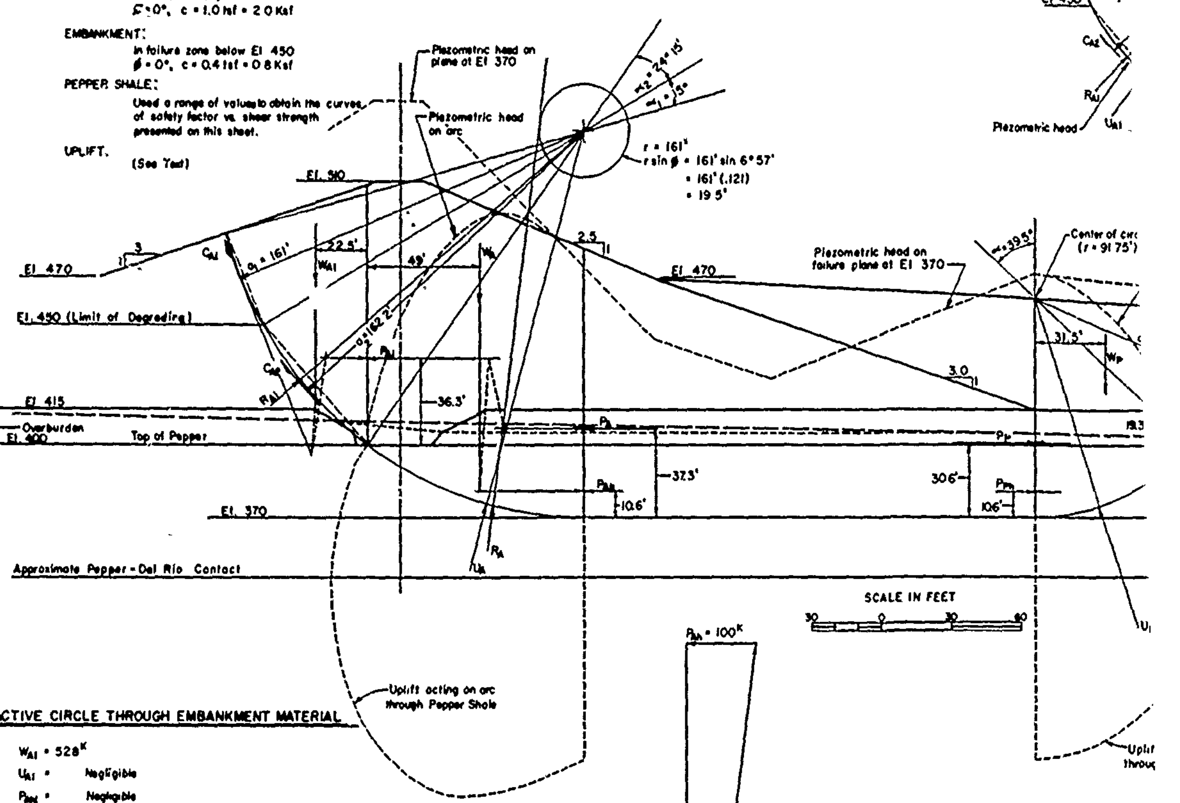
SHEAR STRENGTH:

EMBANKMENT:
New Construction above El. 450, berms and overburden,
$\phi = 0°$, c = 1.0 tsf = 2.0 Ksf

EMBANKMENT:
In failure zone below El. 450
$\phi = 0°$, c = 0.4 tsf = 0.8 Ksf

PEPPER SHALE:
Used a range of values to obtain the curves of safety factor vs. shear strength presented on this sheet.

UPLIFT.
(See Text)



ACTIVE CIRCLE THROUGH EMBANKMENT MATERIAL

W_{A1} = 528^K
U_{A1} = Negligible
P_{AA1} = Negligible

$L_a = r\alpha_1 = 161\left(\frac{15}{57.3}\right) = 42.2'$

$L_c = 2r \sin\frac{\alpha_1}{2} = 2(161)\sin\left(\frac{15°}{2}\right) = 322(.131) = 42.2'$

$a_1 = r\left(\frac{L_a}{L_c}\right) = 161\left(\frac{42.2}{42.2}\right) = 161'$

$C_{A1} = \frac{cL_c}{SF} = \frac{2.0(42.2)}{SF} = \frac{84.4^K}{SF}$

$L_a = r\alpha_2 = 161\left(\frac{24.25}{57.3}\right) = 68.1'$

$L_c = 2r \sin\frac{\alpha_2}{2} = 2(161)\sin\left(\frac{24.25°}{2}\right) = 322(.210) = 67.6'$

$a_2 = r\left(\frac{L_a}{L_c}\right) = 161\left(\frac{68.1}{67.6}\right) = 162.2'$

$C_{A2} = \frac{cL_c}{SF} = \frac{0.8(67.6)}{SF} = \frac{54.1^K}{SF}$

P_{A1} = determined from force polygon No. 1

ACTIVE CIRCLE THROUGH PEPPER SHALE

W_A = 1473^K
U_A = 584^K
P_{AA} = 100^K
P_{A1} = determined from force polygon No. 1
P_A = determined from force polygon No. 2

PASSIVE CIRCLE I THROUGH EMBANKMENT AND OVE

W_{PI} = 121^K
U_{PI} = Negligible
P_{PAI} = Negligible
P_{PI} = determined from force polygon (not shown)

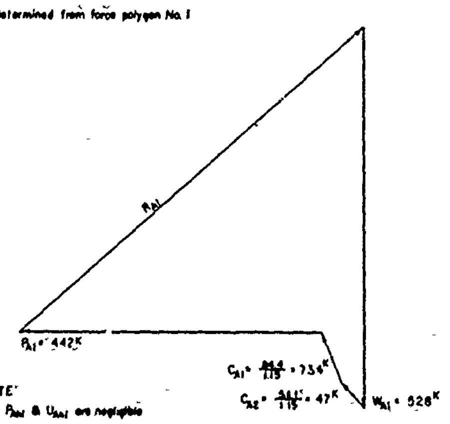
$L_a = r\alpha = 9$
$L_c = 2r \sin$
$a = r\left(\frac{L_a}{L_c}\right)$
$C = \frac{cL_c}{SF}$

PASSIVE CIRCLE I THROUGH PEPPER SHALE

W_P = 648^K
U_P = 309^K
P_{PA} = 100^K
P_{PI} = determined from force polygon for arc segment through emban and overburden material (not shown)
P_P = determined from force polygon for arc segment through Pepper Sh

SLIDING BLOCK FOR PASSIVE CIRCLE I

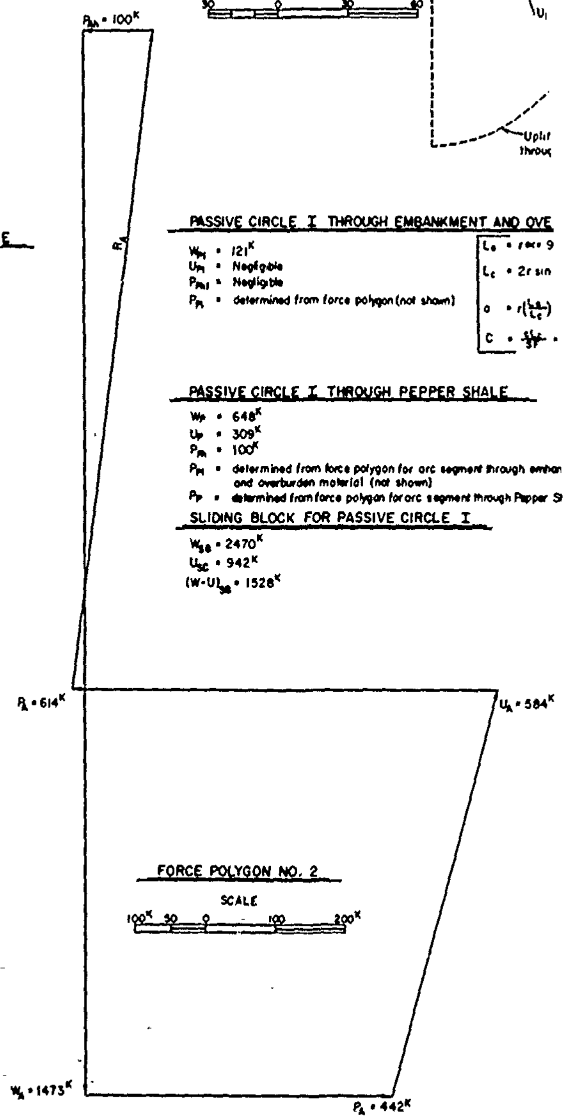
W_{SB} = 2470^K
U_{SC} = 942^K
$(W-U)_{SB}$ = 1528^K



NOTE:
P_{AA1} & U_{AA1} are negligible

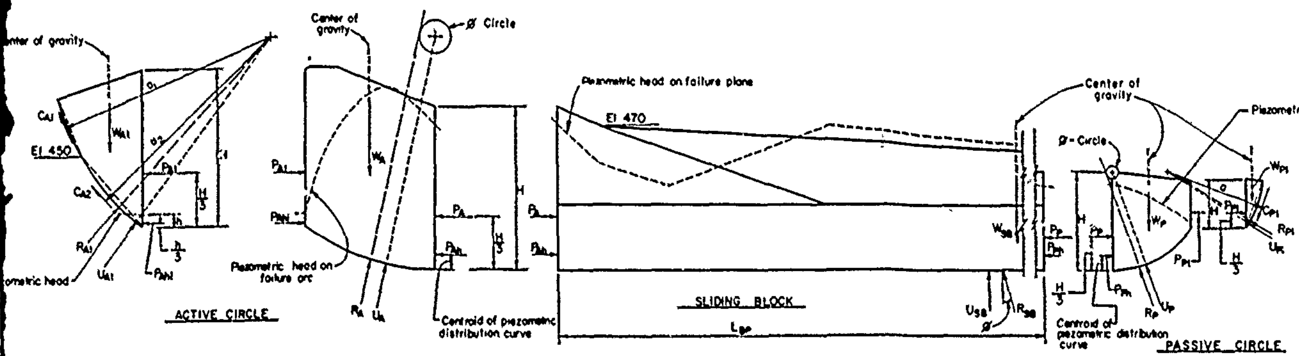
FORCE POLYGON NO. 1

SCALE



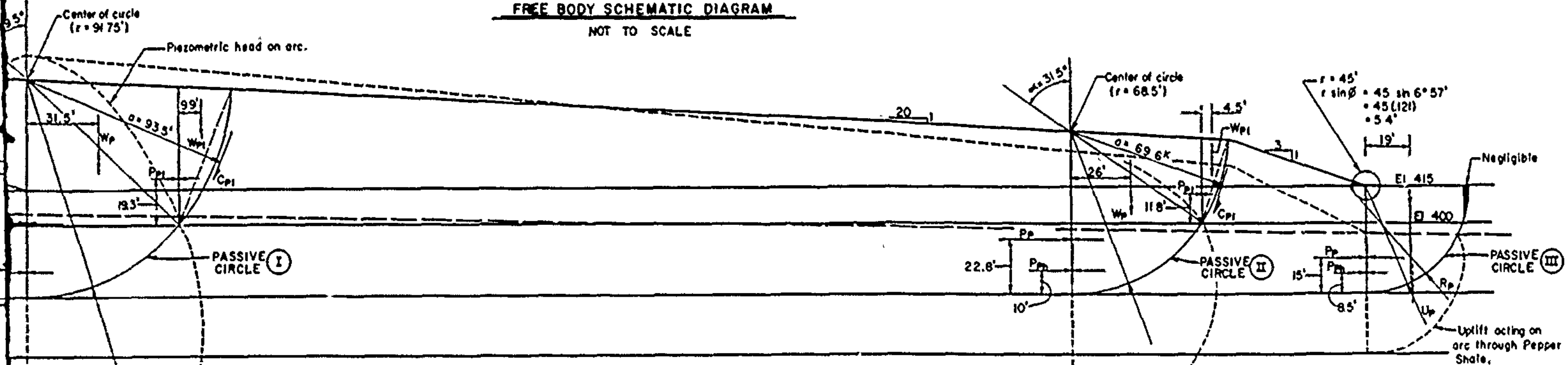
FORCE POLYGON NO. 2

SCALE



FREE BODY SCHEMATIC DIAGRAM

NOT TO SCALE



Uplift acting on arc through Pepper Shale

Uplift acting on arc through Pepper Shale

PASSIVE CIRCLE II THROUGH PEPPER SHALE

$W_P = 429^K$

$U_P = 154^K$

$P_{Pn} = 113^K$

P_{Pt} = determined from force polygon for arc segment through embankment and overburden material (not shown)

P_P = determined from force polygon for arc segment through Pepper Shale (not shown)

SLIDING BLOCK FOR PASSIVE CIRCLE II

$W_{SB} = 7230^K$

$U_{SB} = 3241^K$

$(W-U)_{SB} = 3989^K$

PASSIVE CIRCLE III

$W_P = 206^K$

$U_P = 50^K$

$P_{Pn} = 20^K$

P_P = determined from force polygon No. 3

PASSIVE CIRCLE II THROUGH EMBANKMENT & OVERBURDEN MATERIAL

$W_{Pt} = 37^K$

$U_{Pt} = 0$

$P_{Pnt} = 0$

$L_a = r\alpha = 68.5\left(\frac{31.5}{57.3}\right) = 37.7'$

$L_c = 2r \sin\frac{\alpha}{2} = 2(68.5)\sin\left(\frac{31.5}{2}\right) = 137(.271) = 37.1'$

$a = r\left(\frac{L_a}{L_c}\right) = 68.5\left(\frac{37.7}{37.1}\right) = 69.6'$

$C = \frac{cL_c}{SF} = \frac{2.0(37.1)}{SF} = \frac{74.2^K}{SF}$

P_{Pt} = determined from force polygon for arc segment through embankment and overburden material (not shown)

SLIDING BLOCK FOR PASSIVE CIRCLE III

$W_{SB} = 8249^K$

$U_{SB} = 3632^K$

$(W-U)_{SB} = 4617^K$

Reference (1) Plate 58; for nomenclature

(2) Plate 57; for pore pressure diagram

(3) Text, Vol. I Appendix C; for procedures [Intermediate resultants (r_1, r_2, r_3, etc.) are not shown for clarity of presentation]

BANKMENT AND OVERBURDEN MATERIAL

$L_a = r\alpha = 91.75\left(\frac{39.5}{57.3}\right) = 63.2'$

$L_c = 2r \sin\frac{\alpha}{2} = 2(91.75)\sin\left(\frac{39.5}{2}\right) = 183.5(.338) = 62.0'$

$a = r\left(\frac{L_a}{L_c}\right) = 91.75\left(\frac{63.2}{62.0}\right) = 93.5'$

$C = \frac{cL_c}{SF} = \frac{2.0(62.0)}{SF} = \frac{124^K}{SF}$

shown)

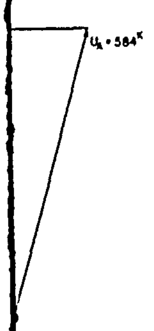
PPER SHALE

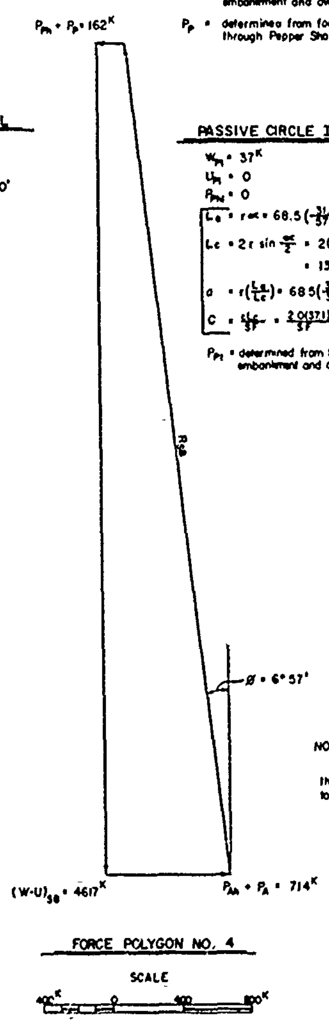
arc segment through embankment

n)

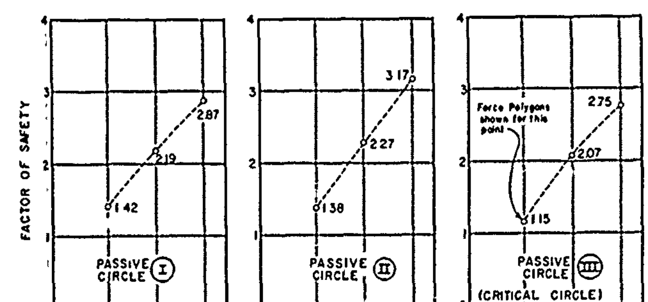
segment through Pepper Shale (not shown)

CLE I





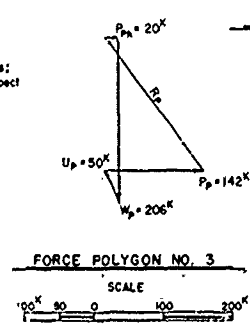
FORCE POLYGON NO. 4



SHEAR STRENGTH OF PEPPER SHALE - Ø effective

NOTE (Force Polygon No 4)

Trail SF = 1.15; Polygon closes; therefore, the safety factor with respect to shear strength is 1.15



FORCE POLYGON NO. 3

BRAZOS RIVER AND TRIBUTARIES, TEXAS

WACO DAM

BOSQUE RIVER, TEXAS

STABILITY ANALYSIS OF REPAIR SECTION FOR EXPECTED PORE PRESSURES

PHI - CIRCLE METHOD

EMBANKMENT SLIDE AREA - STATION 55+00

SCALES AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH JAN. 1963

DESIGN DATA

UNIT WEIGHT SOIL = 130 pcf

UNIT WEIGHT WATER = 62.5 pcf

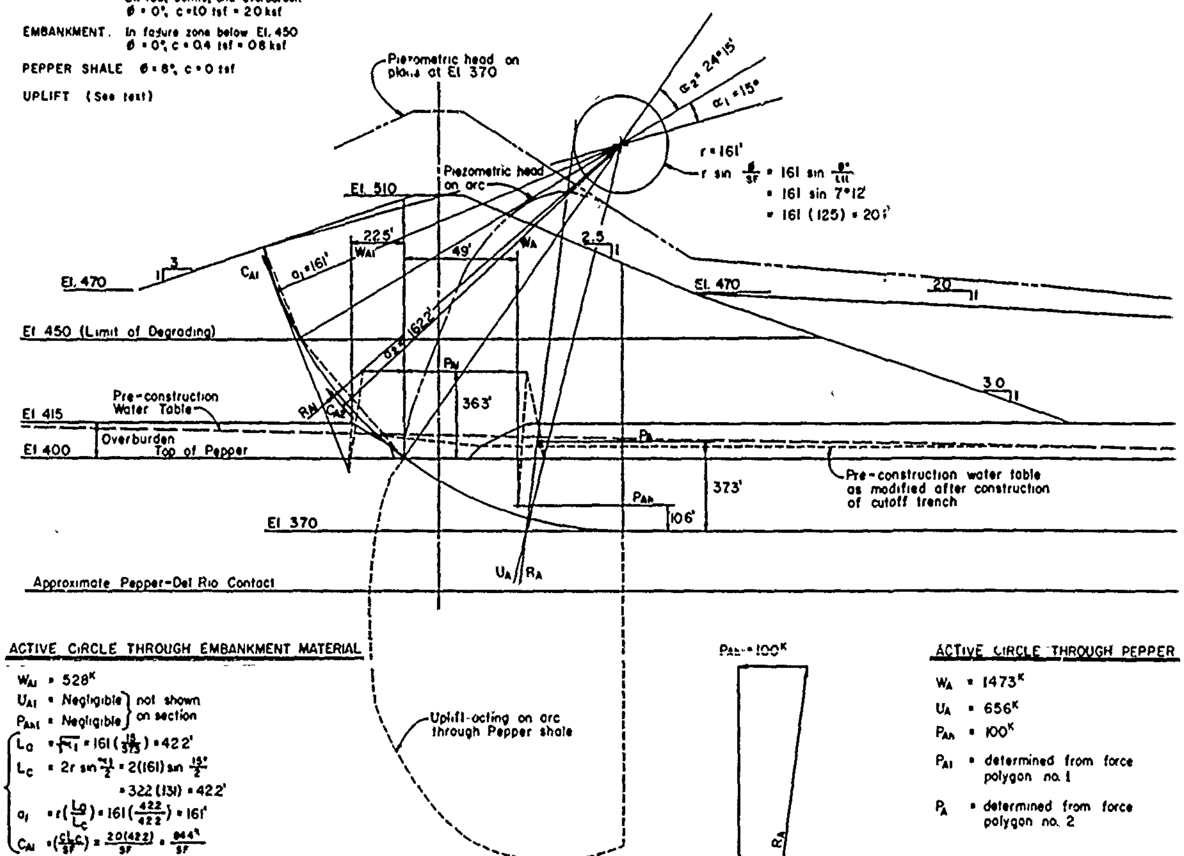
SHEAR STRENGTH:

EMBANKMENT: New construction above El. 450, berms, and overburden $\phi = 0°$, $c = 1.0$ tsf $= 2.0$ ksf

EMBANKMENT. In failure zone below El. 450 $\phi = 0°$, $c = 0.4$ tsf $= 0.8$ ksf

PEPPER SHALE $\phi = 8°$, $c = 0$ tsf

UPLIFT (See text)



ACTIVE CIRCLE THROUGH EMBANKMENT MATERIAL

$W_{A1} = 528^K$

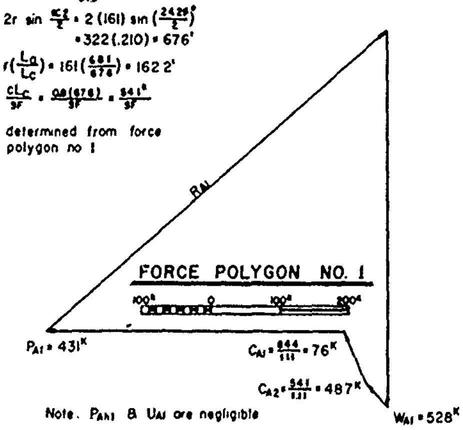
U_{A1} = Negligible } not shown

P_{An1} = Negligible } on section

$$\begin{cases} L_a = r\alpha_1 = 161\left(\frac{15}{57.3}\right) = 42.2' \\ L_c = 2r \sin\frac{\alpha_1}{2} = 2(161) \sin\frac{15°}{2} \\ \qquad = 322(.131) = 42.2' \\ a_1 = r\left(\frac{L_a}{L_c}\right) = 161\left(\frac{42.2}{42.2}\right) = 161' \\ C_{A1} = \left(\frac{cL_c}{SF}\right) = \frac{2.0(42.2)}{SF} = \frac{84.4^k}{SF} \end{cases}$$

$$\begin{cases} L_a = r\alpha_2 = 161\left(\frac{24.25°}{57.3}\right) = 68.1' \\ L_c = 2r \sin\frac{\alpha_2}{2} = 2(161) \sin\left(\frac{24.25°}{2}\right) \\ \qquad = 322(.210) = 67.6' \\ a_2 = r\left(\frac{L_a}{L_c}\right) = 161\left(\frac{68.1}{67.6}\right) = 162.2' \\ C_{A2} = \frac{cL_c}{SF} = \frac{0.8(67.6)}{SF} = \frac{54.1^k}{SF} \end{cases}$$

P_{A1} = determined from force polygon no 1



Note. P_{An1} & U_{A1} are negligible

ACTIVE CIRCLE THROUGH PEPPER

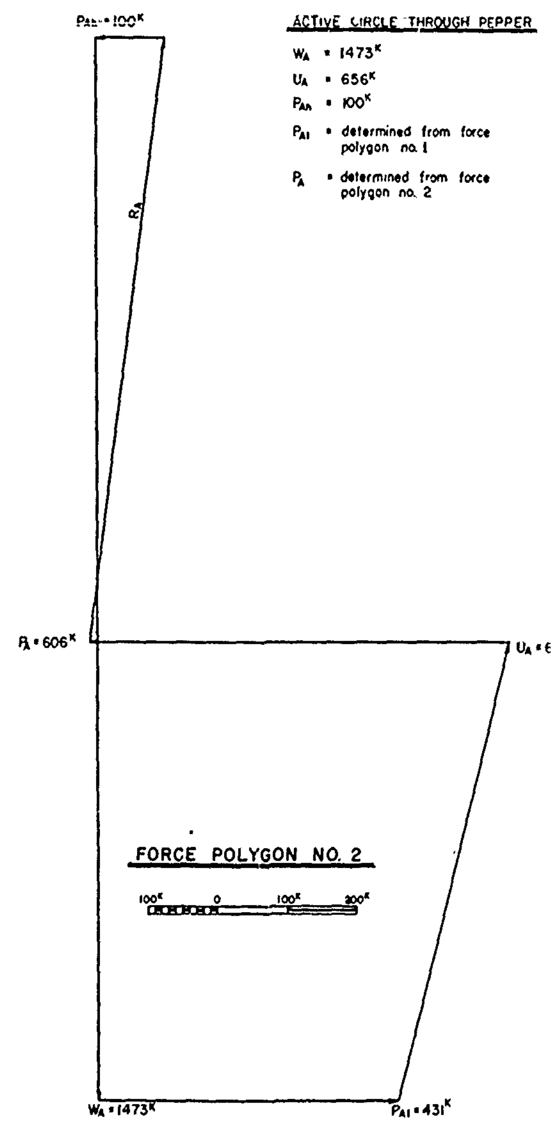
$W_A = 1473^K$

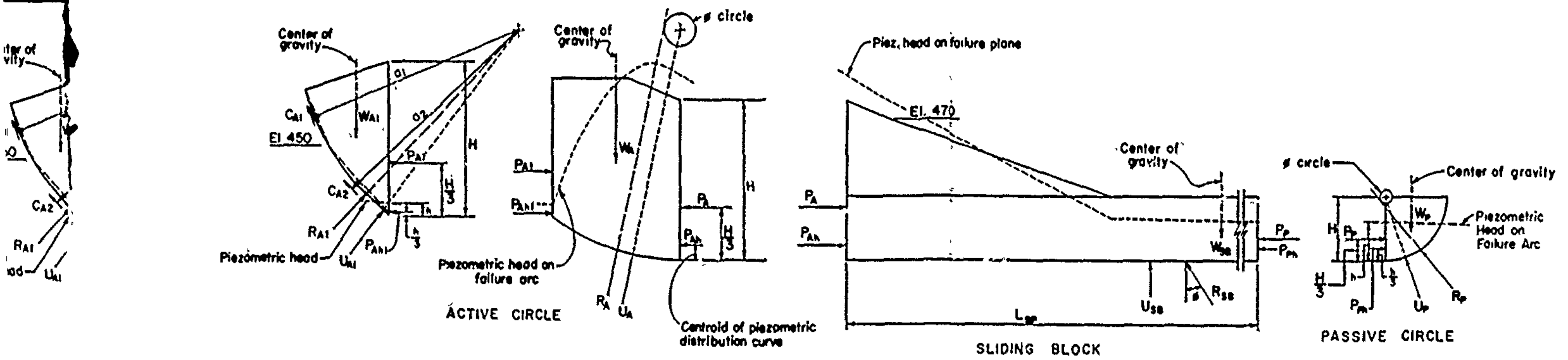
$U_A = 656^K$

$P_{An} = 100^K$

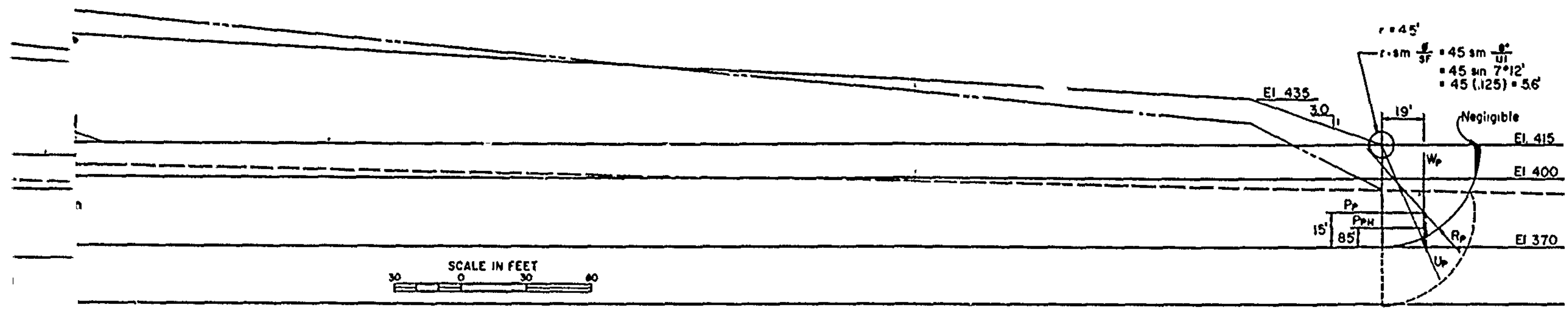
P_{A1} = determined from force polygon no. 1

P_A = determined from force polygon no. 2





FREE BODY SCHEMATIC DIAGRAM
NOT TO SCALE



THROUGH PEPPER SHALE

...d from force ...o. 1

...l from force ...o 2

SLIDING BLOCK

W_{SB} = 8249^K

U_{SB} = 4037^K

$(W-U)_{SB}$ = 4212^K

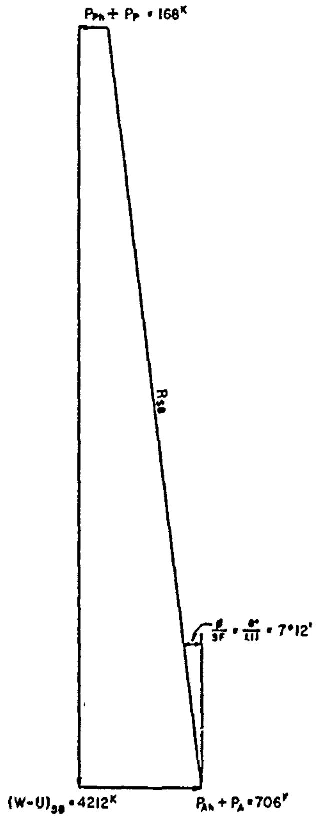
PASSIVE CIRCLE

W_P = 206^K

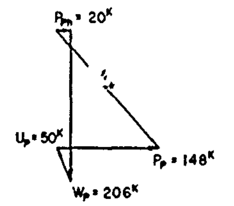
U_P = 50^K

P_{Ph} = 20^K

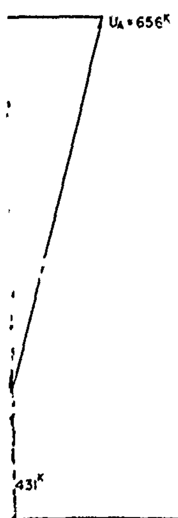
P_P = determined from force polygon no. 3



FORCE POLYGON NO 4



FORCE POLYGON NO 3



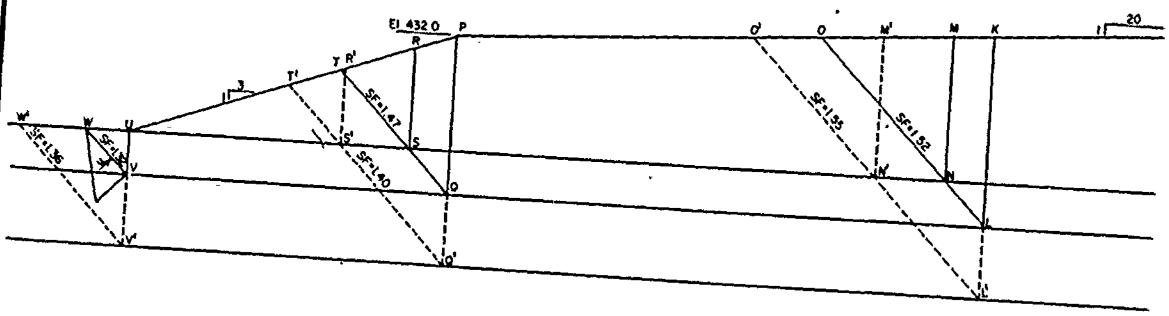
Reference
(1) Plate 58, for nomenclature
(2) Plate 57, for pore pressure diagram
(3) Text, Vol. I, Appendix C, for procedures
[Intermediate resultants (r_1, r_2, r_3, etc.) are not shown for clarity of presentation]

Note: Trial SF = 1.11; Polygon closes; therefore, the safety factor with respect to shear strength is 1.11

BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS
STABILITY ANALYSIS OF REPAIR SECTION
FOR MAXIMUM PORE PRESSURES THAT COULD OCCUR
PHI – CIRCLE METHOD
EMBANKMENT SLIDE AREA – STATION 55+00
SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN. 1963

CORPS OF ENGINEERS



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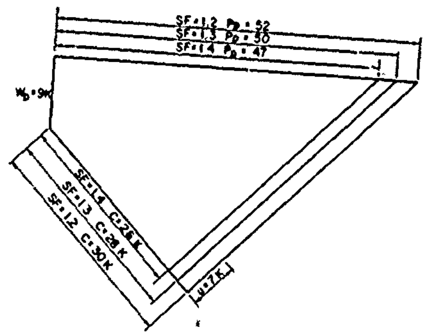
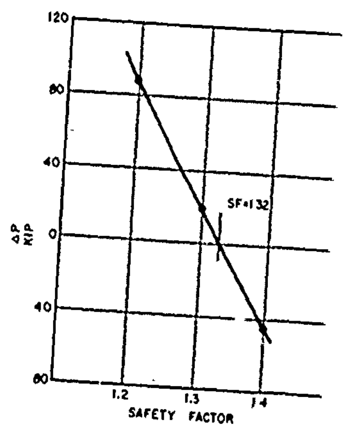
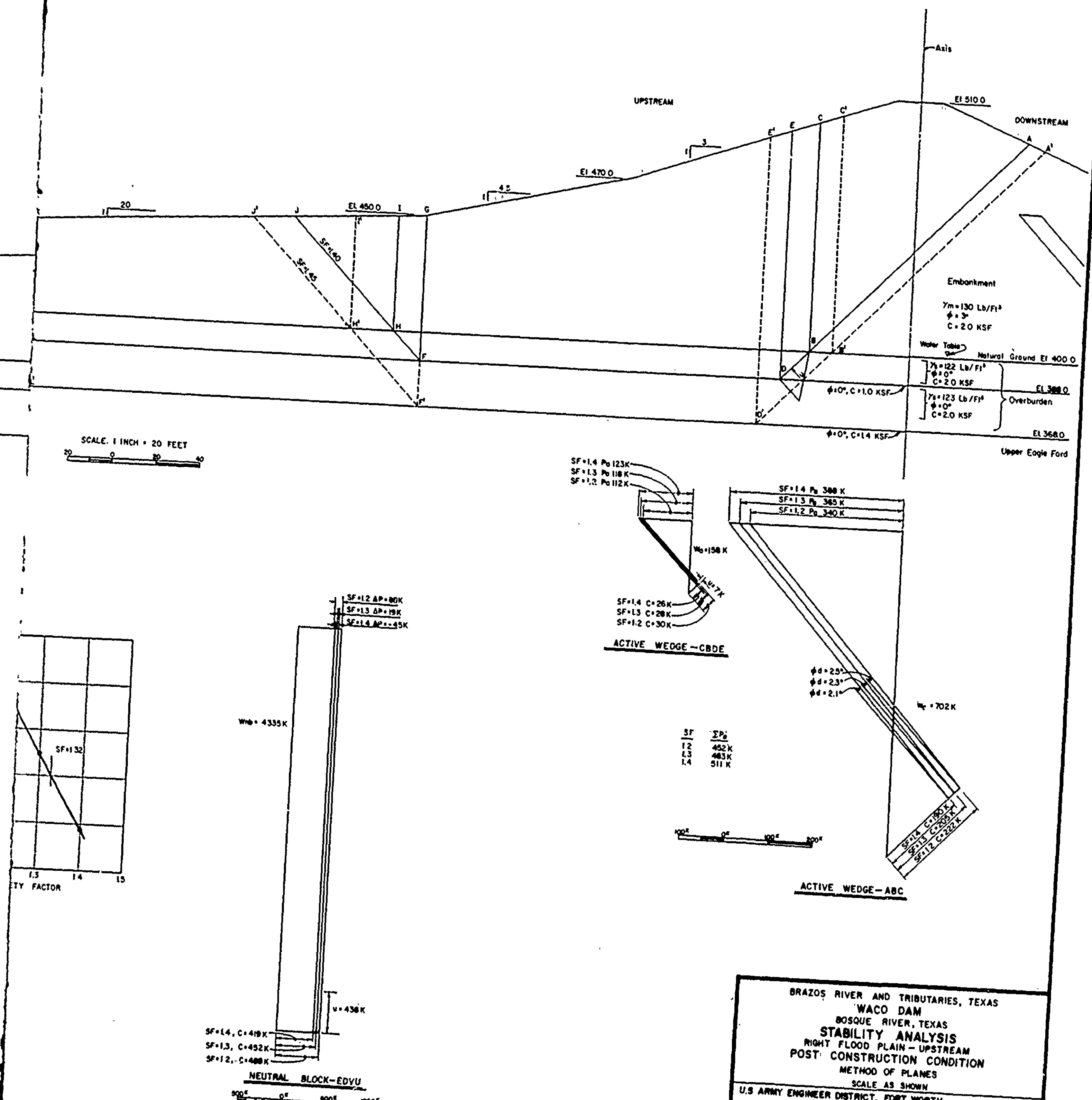


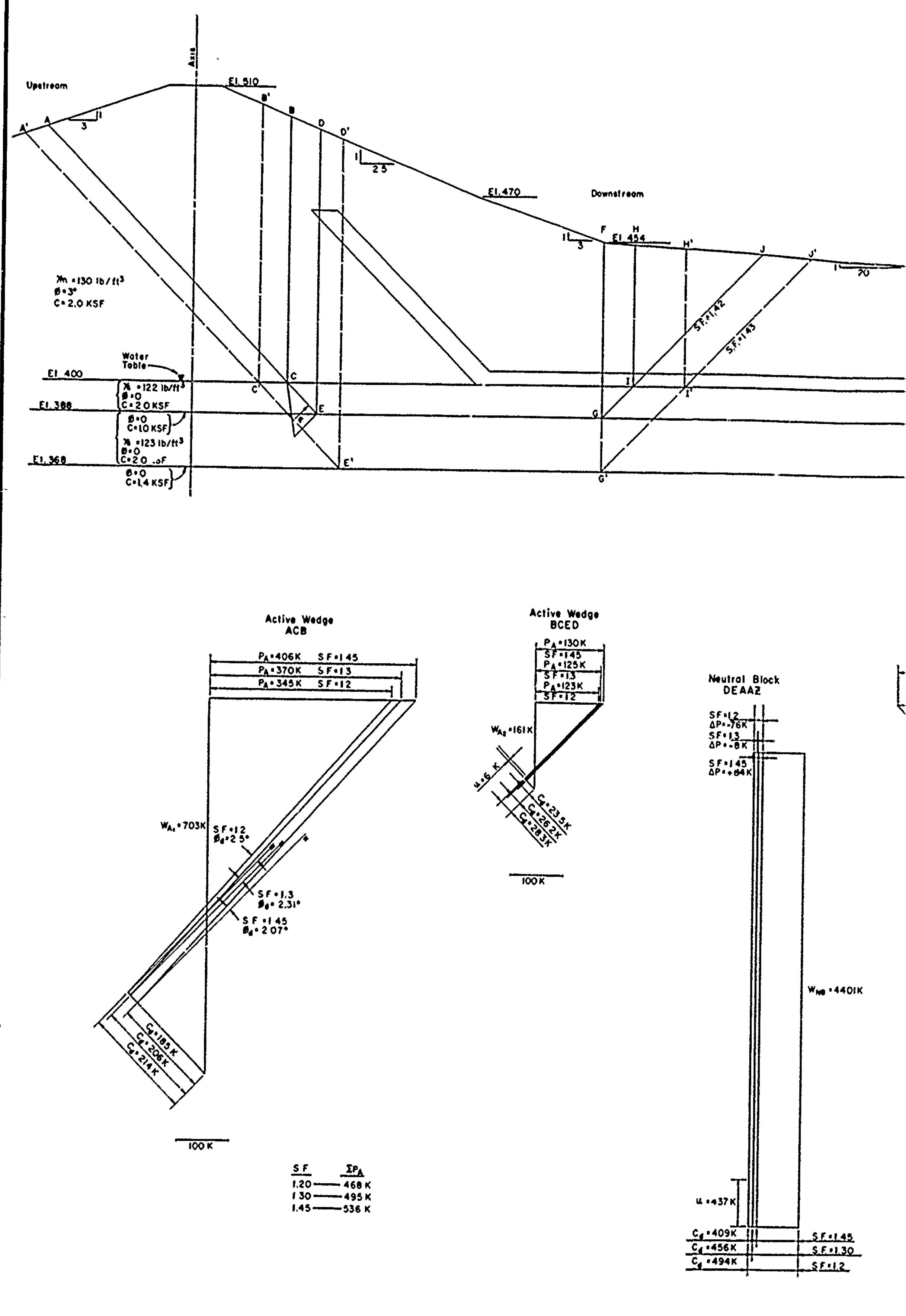
PASSIVE WEDGE — UVW

10K 0K 10K 20K

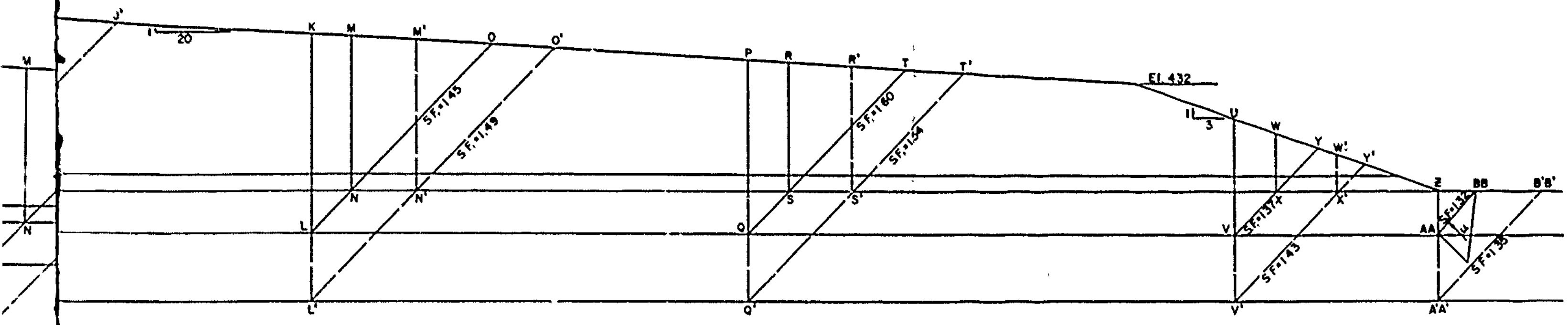


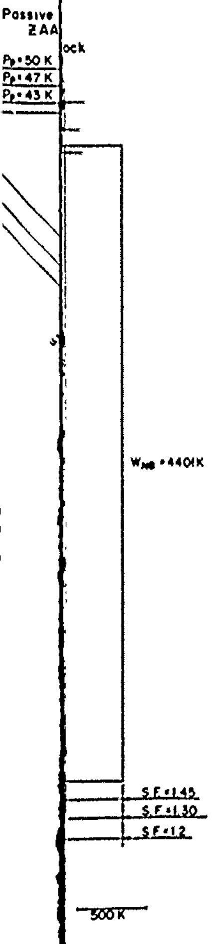


BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS
STABILITY ANALYSIS
RIGHT FLOOD PLAIN - UPSTREAM
POST CONSTRUCTION CONDITION
METHOD OF PLANES
SCALE AS SHOWN
U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN. 1963

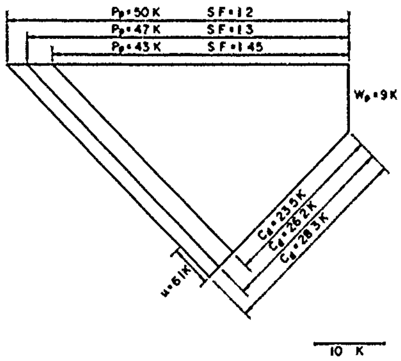


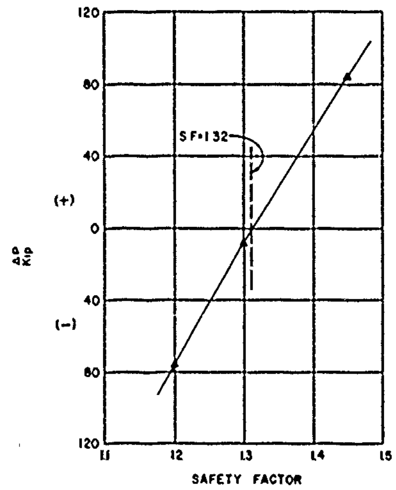
S.F	ΣP_A
1.20	468 K
1.30	495 K
1.45	536 K





Passive Wedge
ZAABB





BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS
STABILITY ANALYSIS
RIGHT FLOOD PLAIN — DOWNSTREAM
POST CONSTRUCTION CONDITION
METHOD OF PLANES
SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN. 1963

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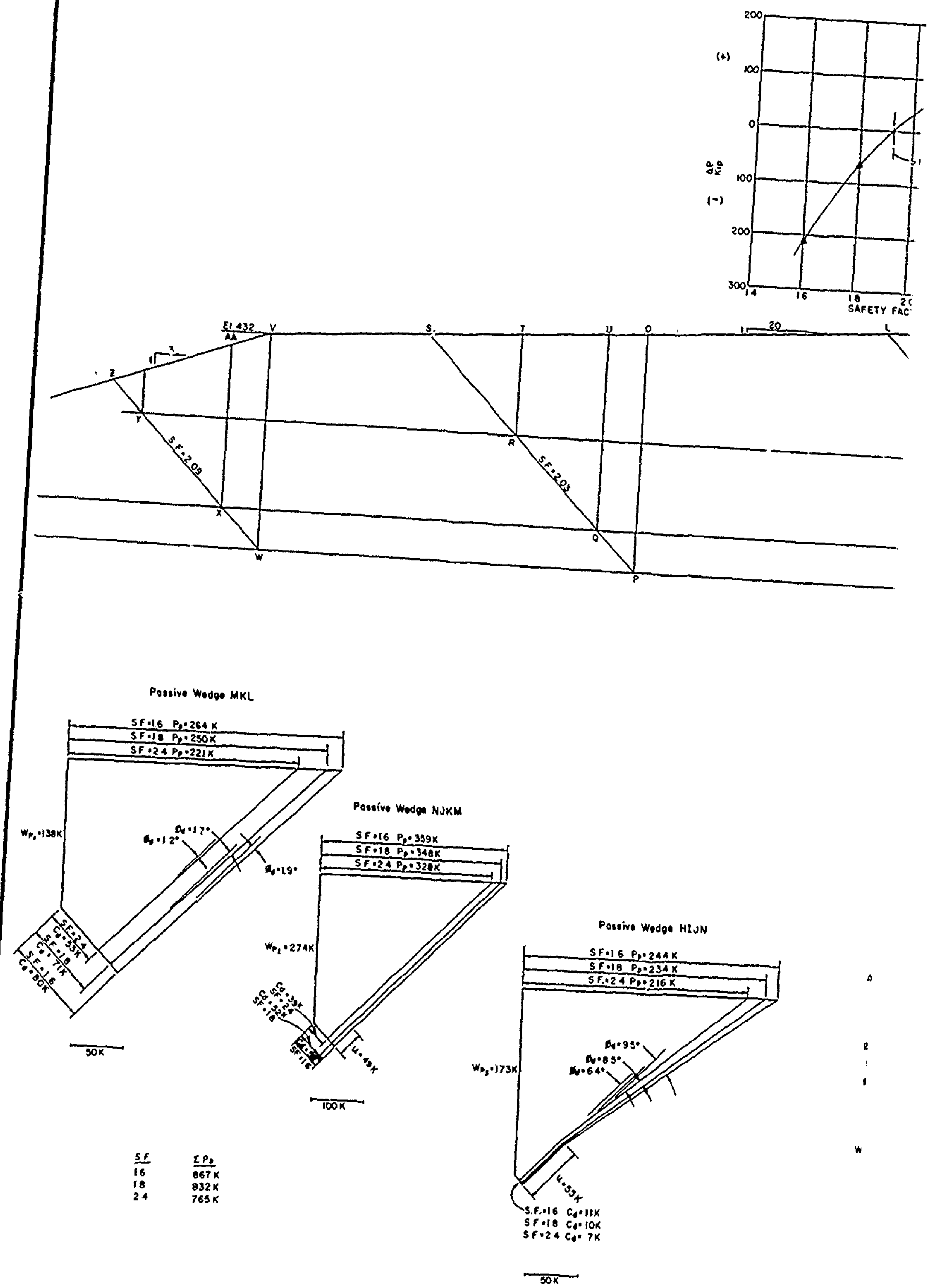
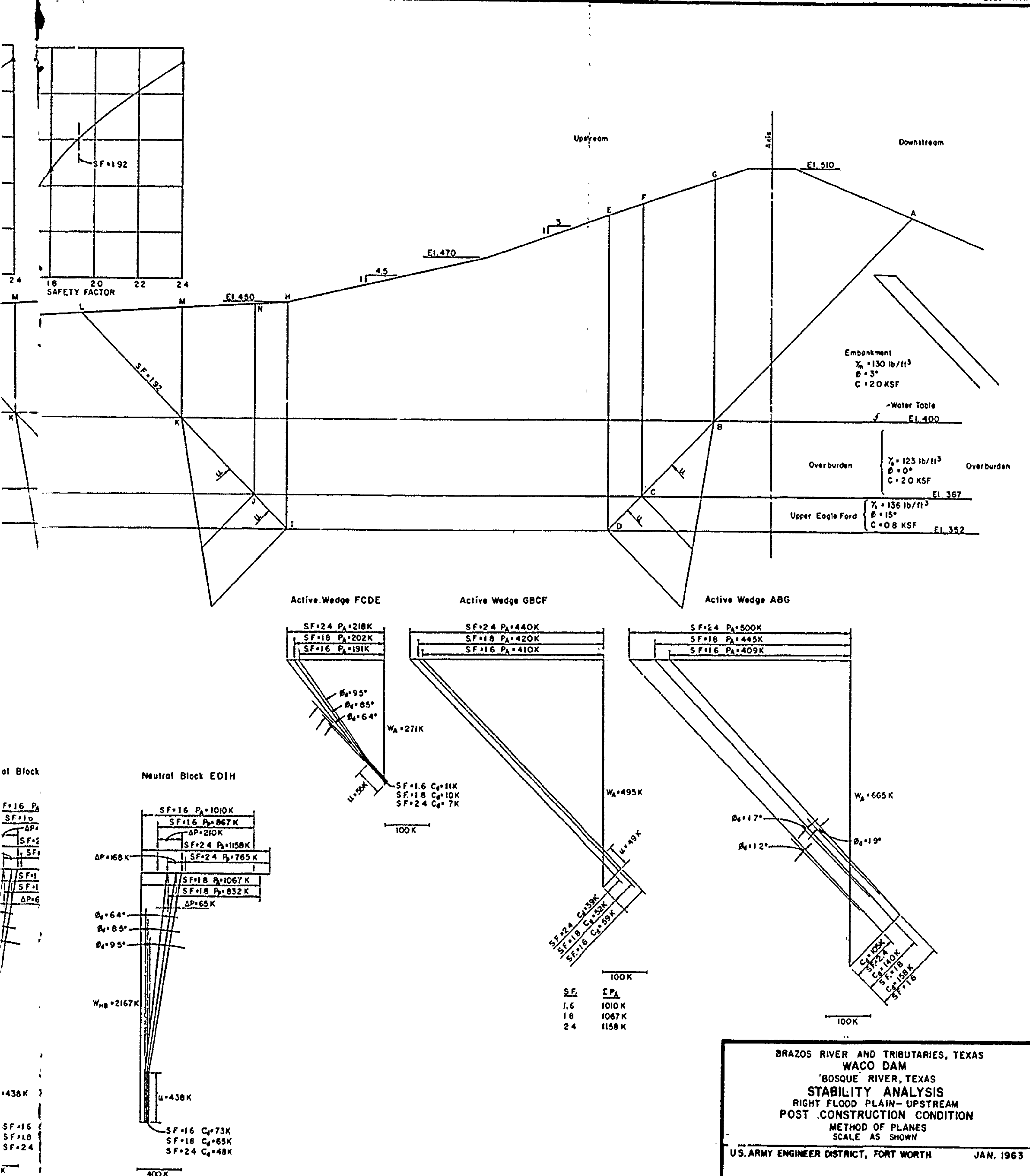


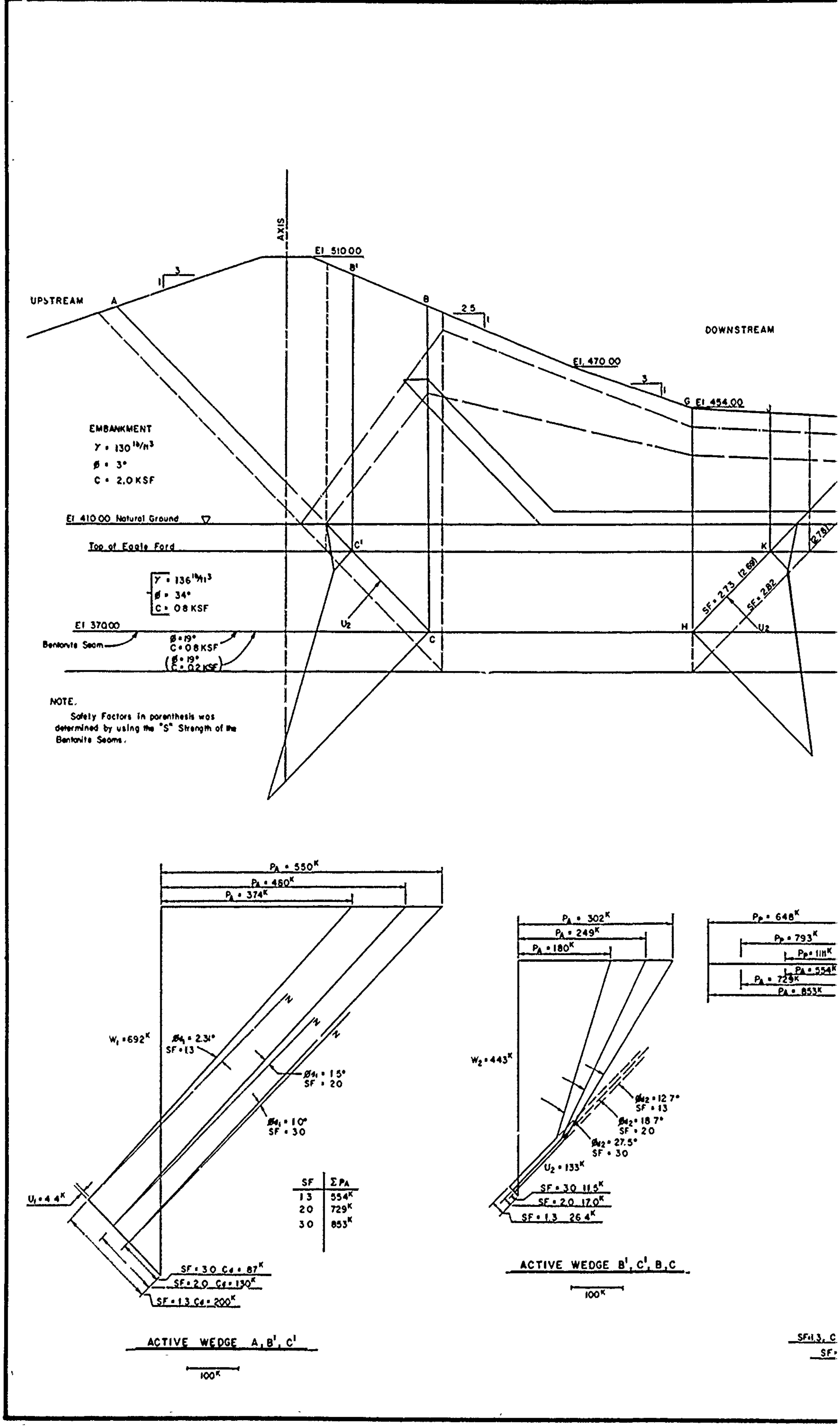
SF	ΣPp
1.6	867 K
1.8	832 K
2.4	765 K



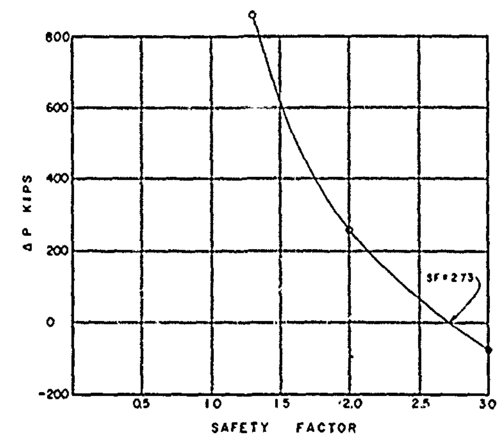
BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS
STABILITY ANALYSIS
RIGHT FLOOD PLAIN- UPSTREAM
POST CONSTRUCTION CONDITION
METHOD OF PLANES
SCALE AS SHOWN

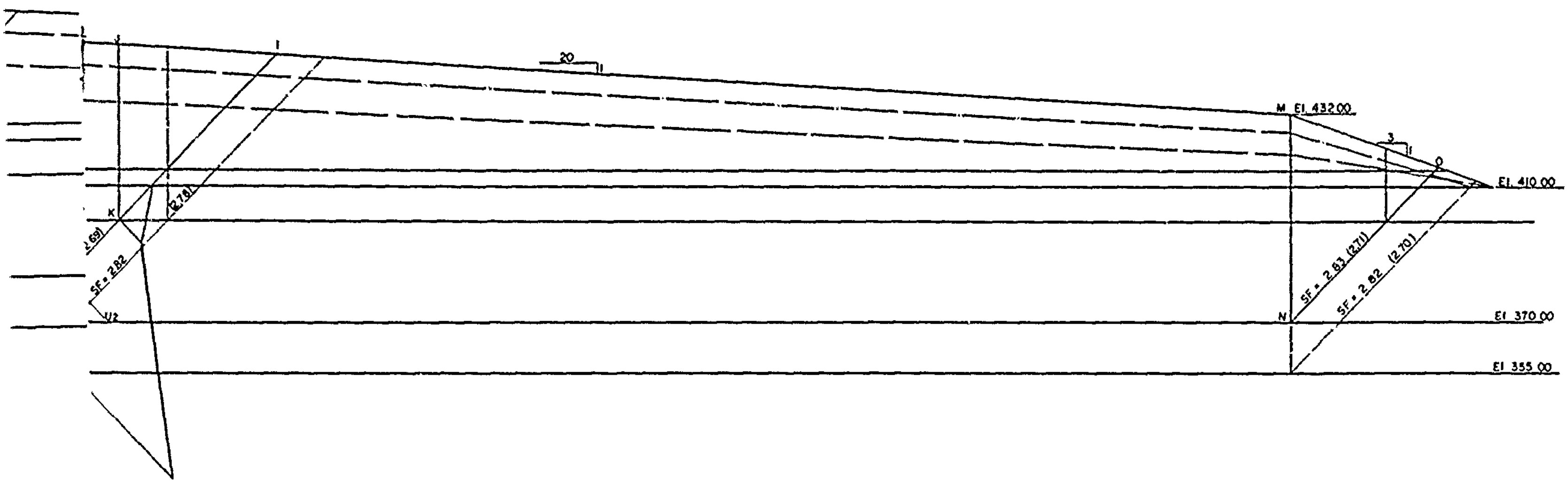
U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN. 1963

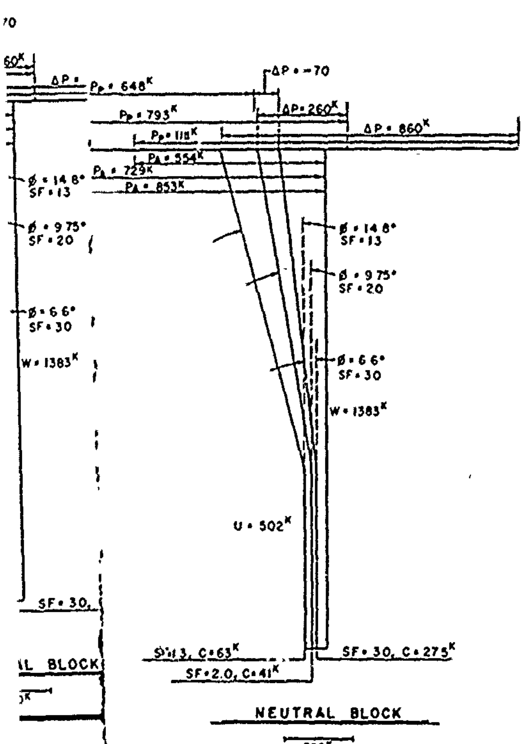
CORPS OF ENGINEERS

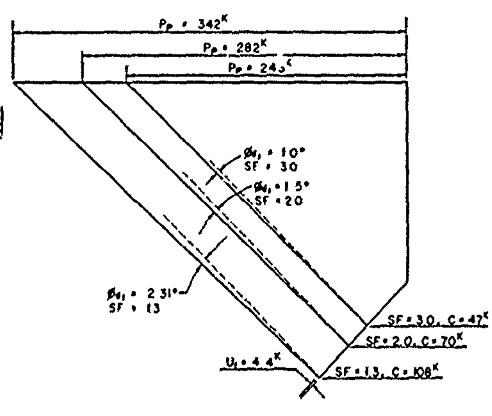


SF	ΣPA
1.3	554K
2.0	729K
3.0	853K



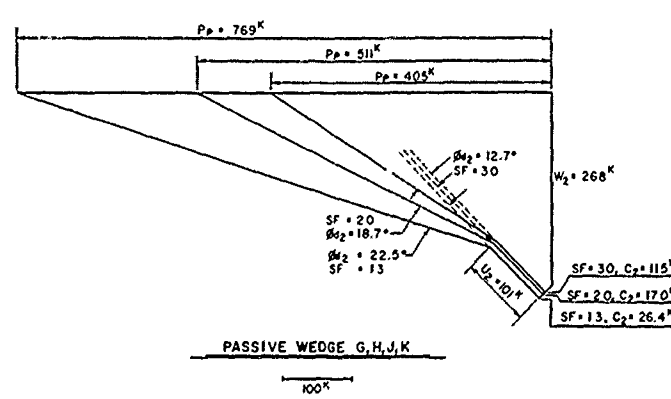






PASSIVE WEDGE J, K, I

60K



PASSIVE WEDGE G, H, J, K

100K

SF	ΣP_P
1.3	1111^K
2.0	793^K
3.0	648^K

BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
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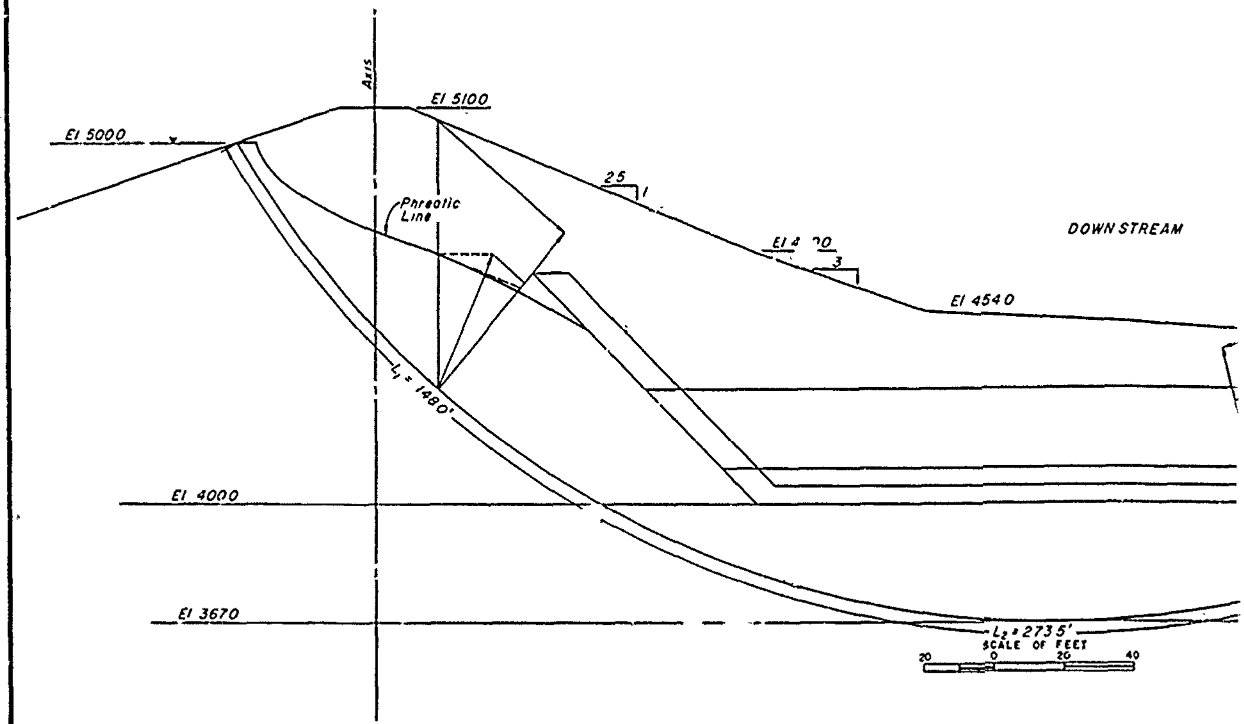
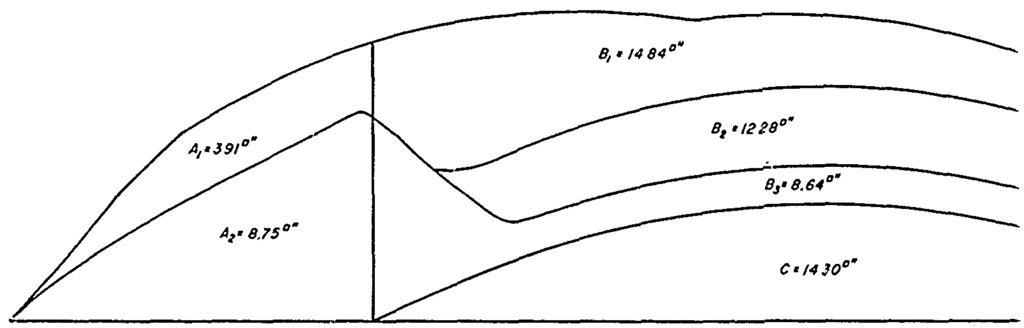
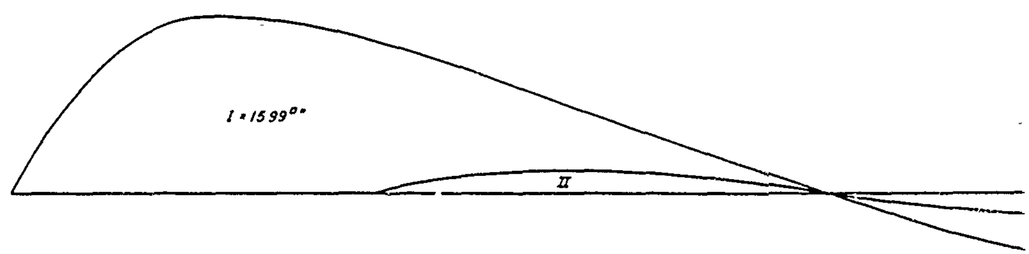
STABILITY ANALYSIS
LEFT FLOOD PLAIN - DOWNSTREAM
POST CONSTRUCTION CONDITION
METHOD OF PLANES
SCALE AS SHOWN
U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN, 1963





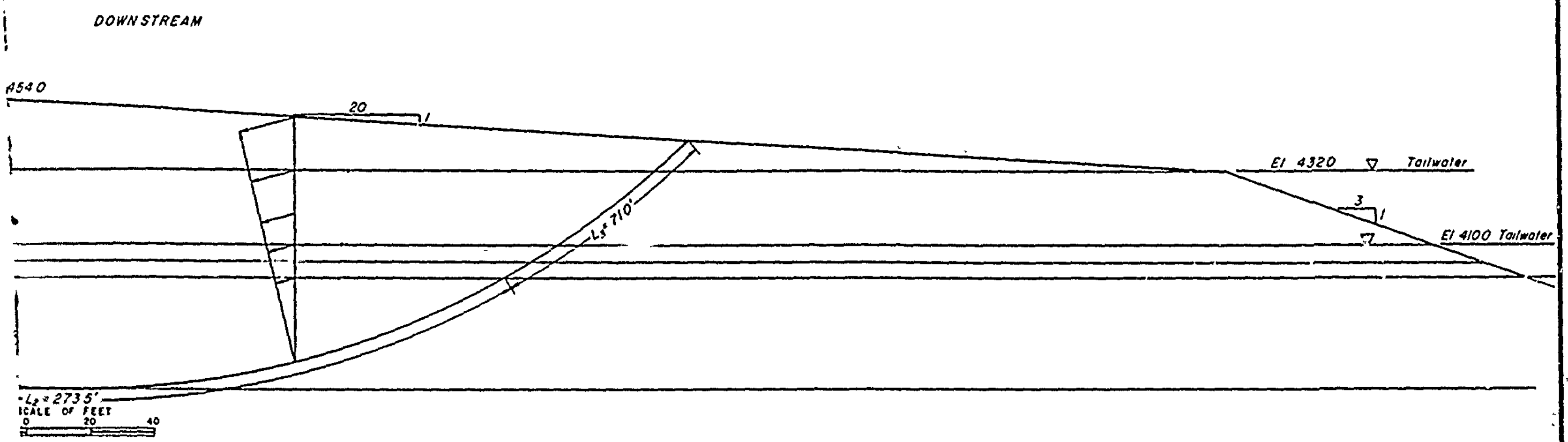
NORMAL STRESS DIAGRAM
Feet

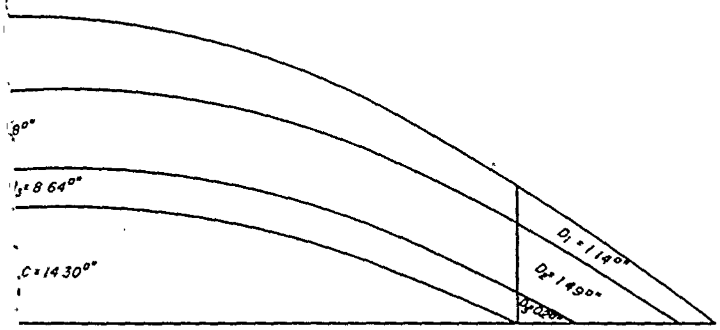


TANGENTIAL STRESS DIAGRAM
Feet

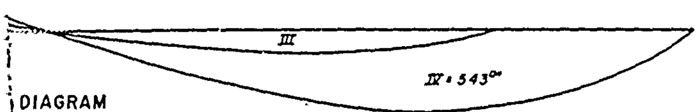
DESIGN DATA

	"R" Strength	"S" Strength	Unit Weigh
EMBANKMENT	ϕ = 12° C = 0.4 TSF	ϕ = 22° C = 0.0 TSF	γ_m = 130.0 P γ_s = 130.0 P
FOUNDATION	ϕ = 15° C = 0.4 TSF	ϕ = 25° C = 0.0 TSF	γ_m = 115.0 P γ_s = 123.0 P





S DIAGRAM



DIAGRAM

STEADY STATE SEEPAGE

RIVER SECTION - TW @ EL 4320

"R" Strengths

Segment	Area in²	Area ft²	Unit Wt PCF	Force Kips	Tan Φ	N Tan Φ Kips	L Ft	c KSF	C Kips	N Tan Φ+C Kips
NORMAL FORCES										
$A_1 + A_3$	1266	5,060	1300	6580						
A_2	-875	3,500	624	-2185						
				4395	0213	935	1480	08	1185	2120
$B_1+B_2+B_3$	3576	14,290	1300	1,8580						
C	1430	5,720	1230	7040						
B_2+B_3+C	-3522	14,100	624	8800						
				1,6820	0213	3580	2735	08	2190	5770
$D_1+D_2+D_3$	283	1,132	1300	1472						
D_2+D_3	-169	676	624	-422						
				1050	0213	224	710	08	568	792
							Σ N Tan Φ + C =			8682

TANGENTIAL FORCES

I	1599	6,390	1300	8300
II - III	00	0		00
IV	-543	2,170	1300	-2820
			ΣT =	5480

$$SF = \frac{\Sigma N \tan \phi + C}{\Sigma T} = \frac{8682}{548.0} = \underline{158}$$

TAILWATER @ EL 4320

"R" Strengths, Right Floodplain - SF = 175

"S" Strengths, Right Floodplain - SF = 1.84 180 Avg.

"R" Strengths, River Section - SF = 158

"S" Strengths, River Section - SF 1.64 1.61 Avg.

TAILWATER @ EL 4100

"R" Strengths, Right Floodplain - SF = 191

"S" Strengths, Right Floodplain - SF = 212 202 Avg

"R" Strengths, River Section - SF = 172

"S" Strengths, River Section - SF = 190 181 Avg

SIGN DATA

"S" Strength	Unit Weight
Φ = 22°	γm = 1300 PCF
C = 00 TSF	γs = 1300 PCF
Φ = 25°	γm = 1150 PCF
C = 00 TSF	γs = 1230 PCF

BRAZOS RIVER AND TRIBUTARIES, TEXAS

WACO DAM

BOSQUE RIVER, TEXAS

STABILITY ANALYSIS

STEADY STATE SEEPAGE CONDITION

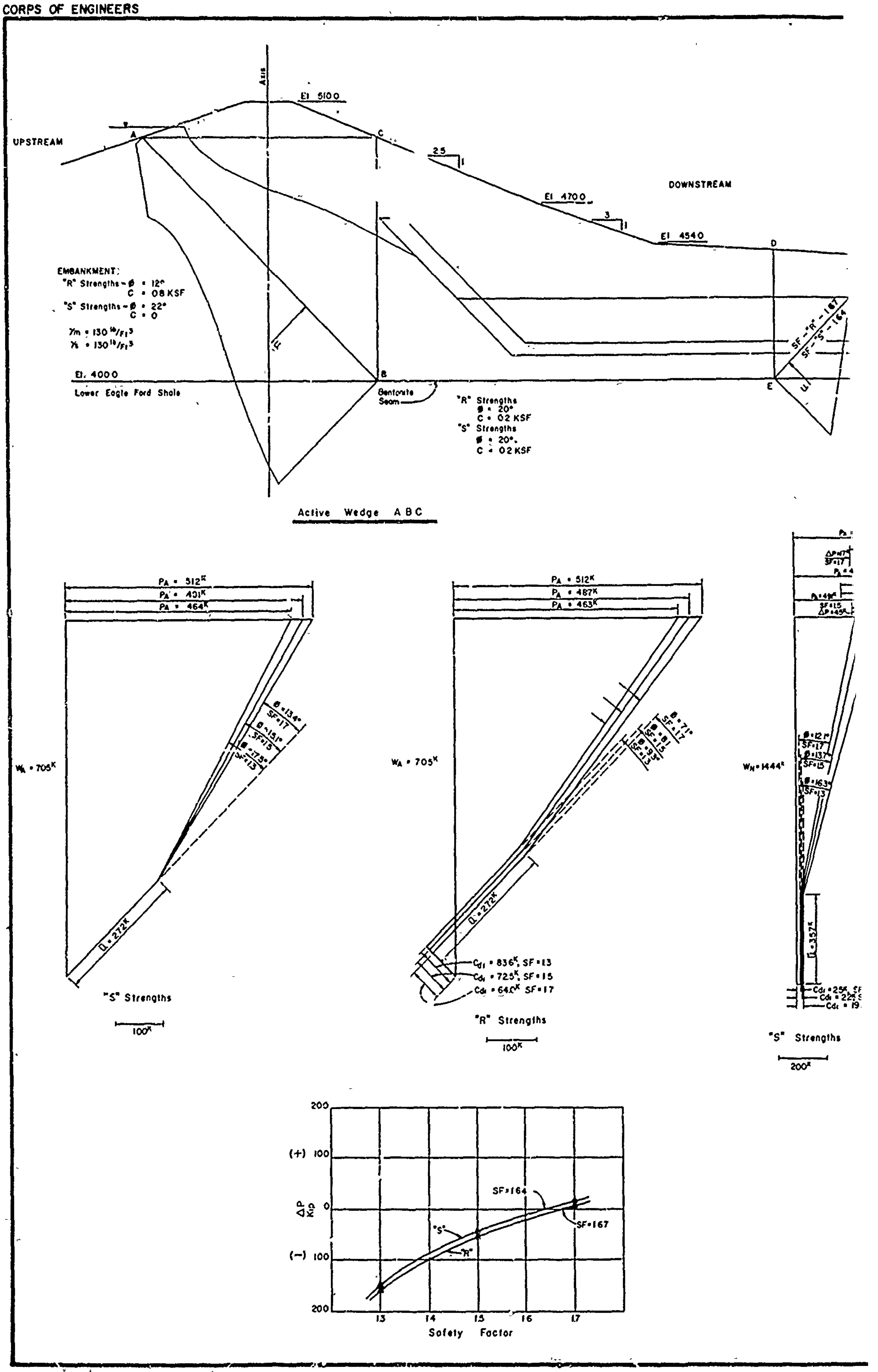
CIRCULAR ARC METHOD

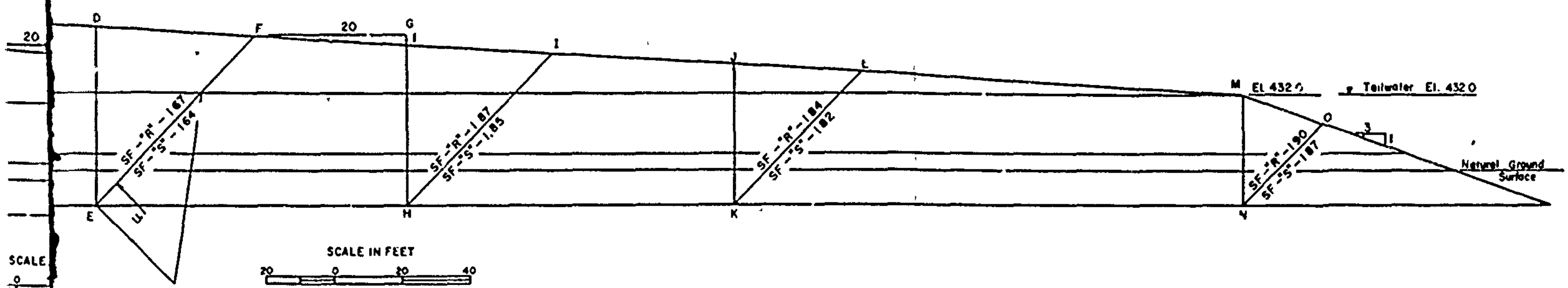
RIGHT FLOODPLAIN AND RIVER SECTION - DOWNSTREAM

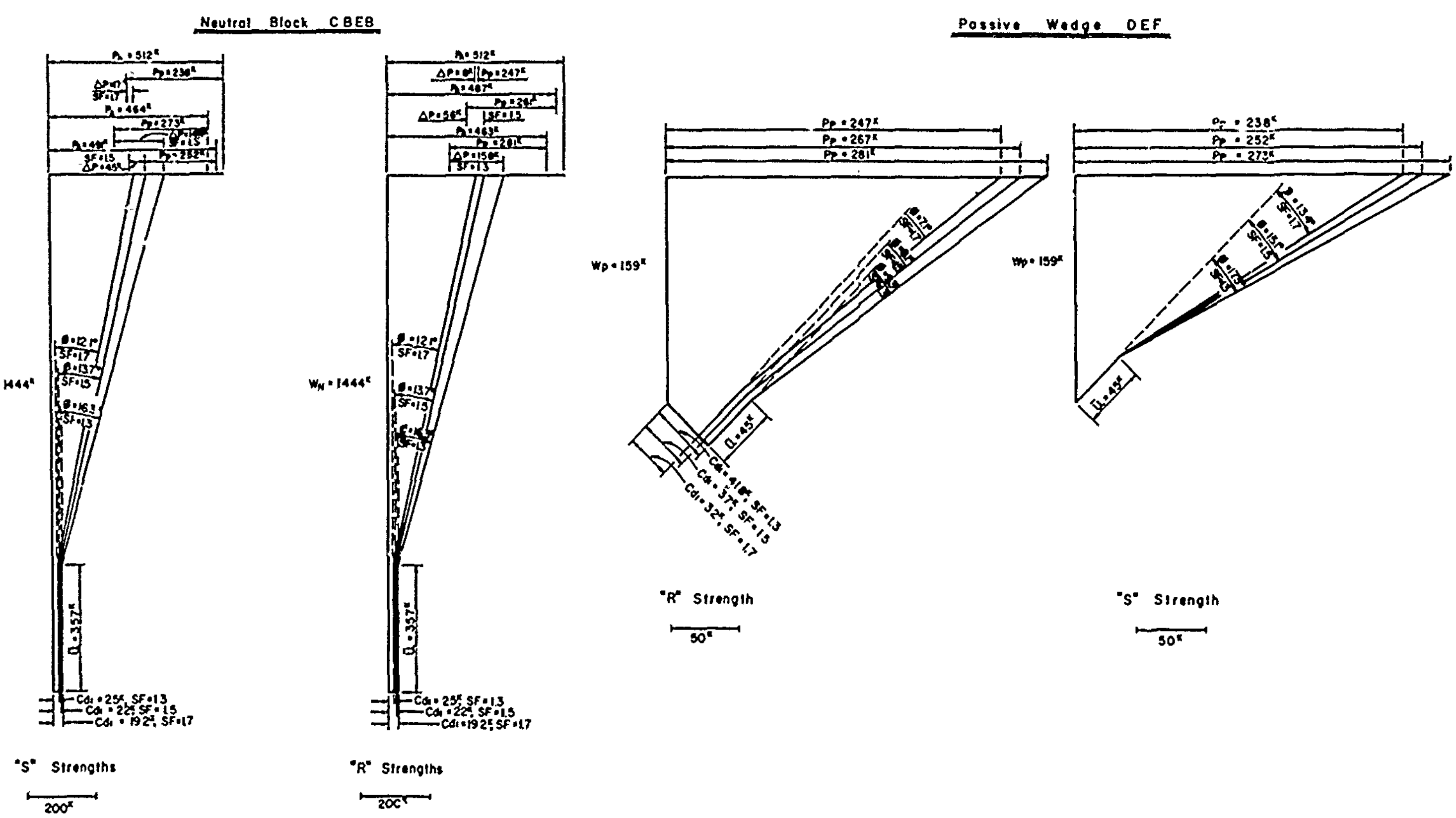
SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH JAN. 1963

CORPS OF ENGINEERS



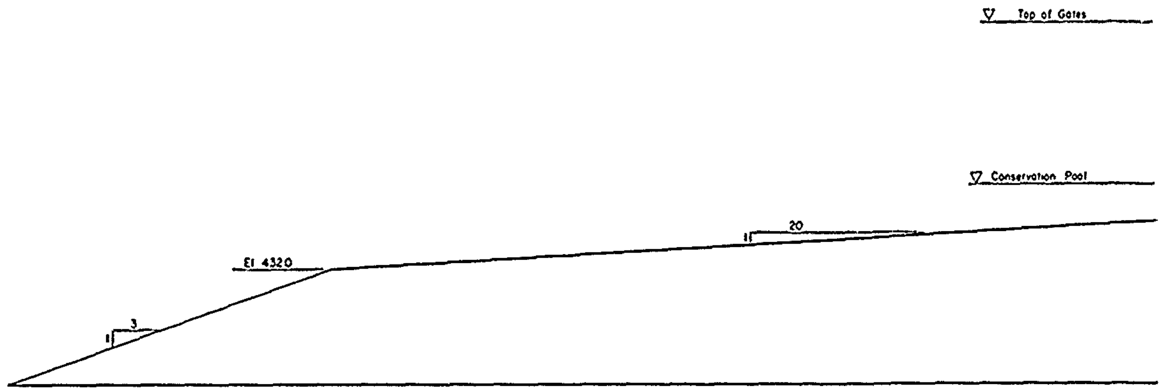




BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS
STABILITY ANALYSIS
LEFT FLOODPLAIN - DOWNSTREAM
STEADY STATE SEEPAGE CONDITION
METHOD OF PLANES
SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH JAN. 1963

CORPS OF ENGINEERS



RAPID DRAWDOWN CONDITION

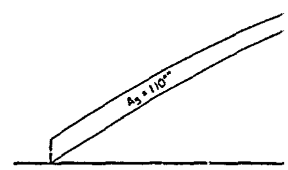
RIGHT FLOODPLAIN SECTION

"R" STRENGTHS

SEGMENT	AREA IN²	AREA FT²	UNIT WT PCF	FORCE KIPS	tan Ø	N tan Ø KIPS	L FT	c KSF	C KIPS	N tan Ø + C KIPS
NORMAL FORCES										
$A_1 + A_3 + A_4$	3789	15,170	130.0	1,970.0						
A_5	110	440	62.4	27.4						
$A_2 + A_3 + A_5$	-36.87	14,750	62.4	- 920.0						
				1,077.4	.213	229.0	369.0	.8	295.0	524.0
							$\Sigma N \tan Ø + C =$			524 KIPS
TANGENTIAL FORCES										
I	10.59	4,235	130.0	550.0						
II	-2.87	1,148	130.0	-149.2						
III	-.75	300	62.4	- 18.7						
				ΣT = 382.1 KIPS						

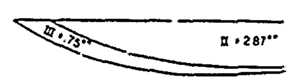
$$SF = \frac{\Sigma N \tan Ø + C}{\Sigma T} = \frac{524 \text{ KIPS}}{382.1 \text{ KIPS}}$$

SF = 1.37

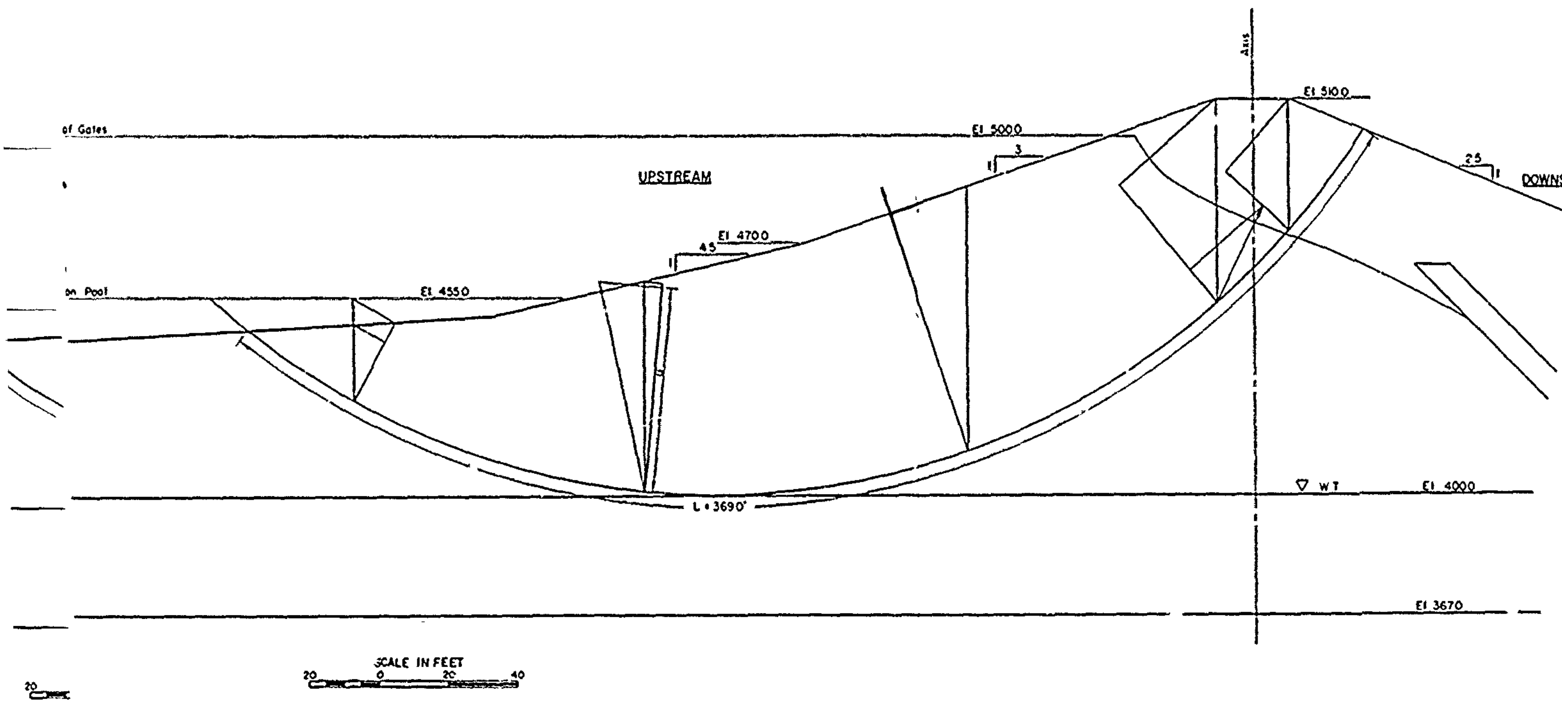


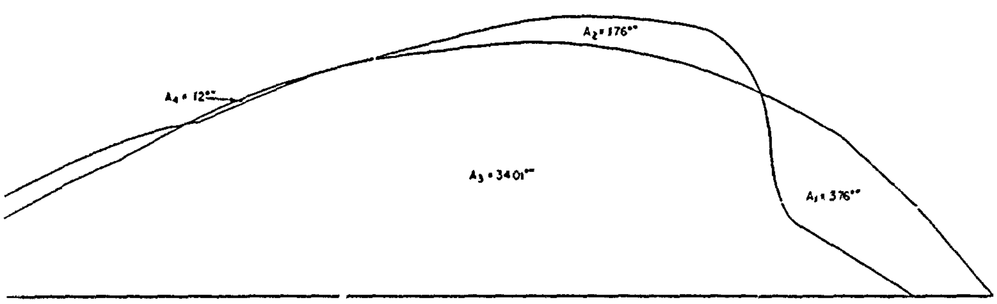
DESIGN DATA

	"R" STRENGTH	UNIT WT
EMBANKMENT	Ø = 12° C = 0.4 TSF	γ_m = 130.0 PCF γ_s = 130.0 PCF

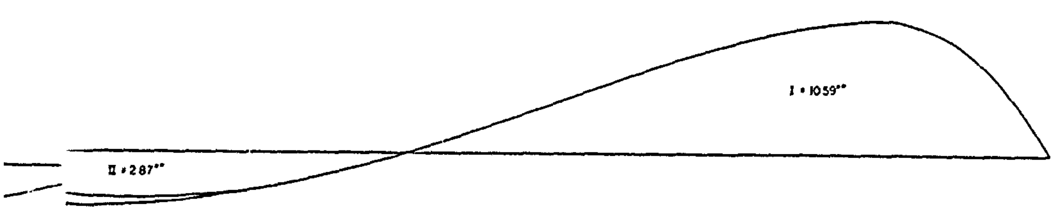


U.S. ARMY





NORMAL STRESS DIAGRAM
FEET

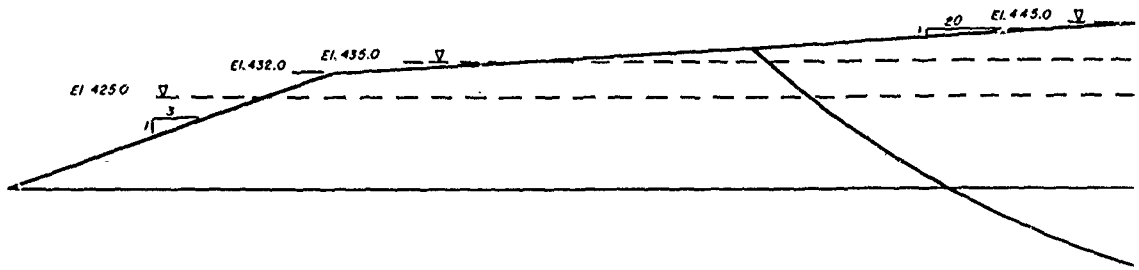


TANGENTIAL STRESS DIAGRAM
FEET

BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS
STABILITY ANALYSIS
RAPID DRAWDOWN CONDITION
CIRCULAR ARC METHOD
RIGHT FLOODPLAIN-UPSTREAM
SCALES AS SHOWN
U.S. ARMY ENGINEER DISTRICT, FORT WORTH JAN. 1963

PLATE 74

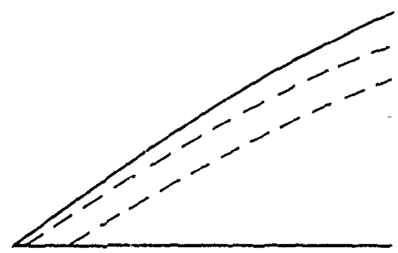
CORPS OF ENGINEERS

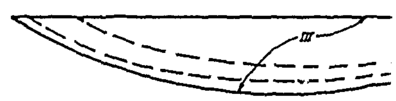


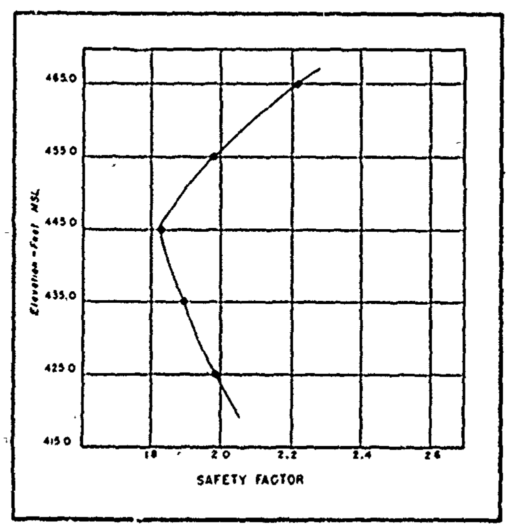
<u>Critical Pool Condition - El. 445.0</u>
<u>"R" Strength</u>
<u>River Section</u>

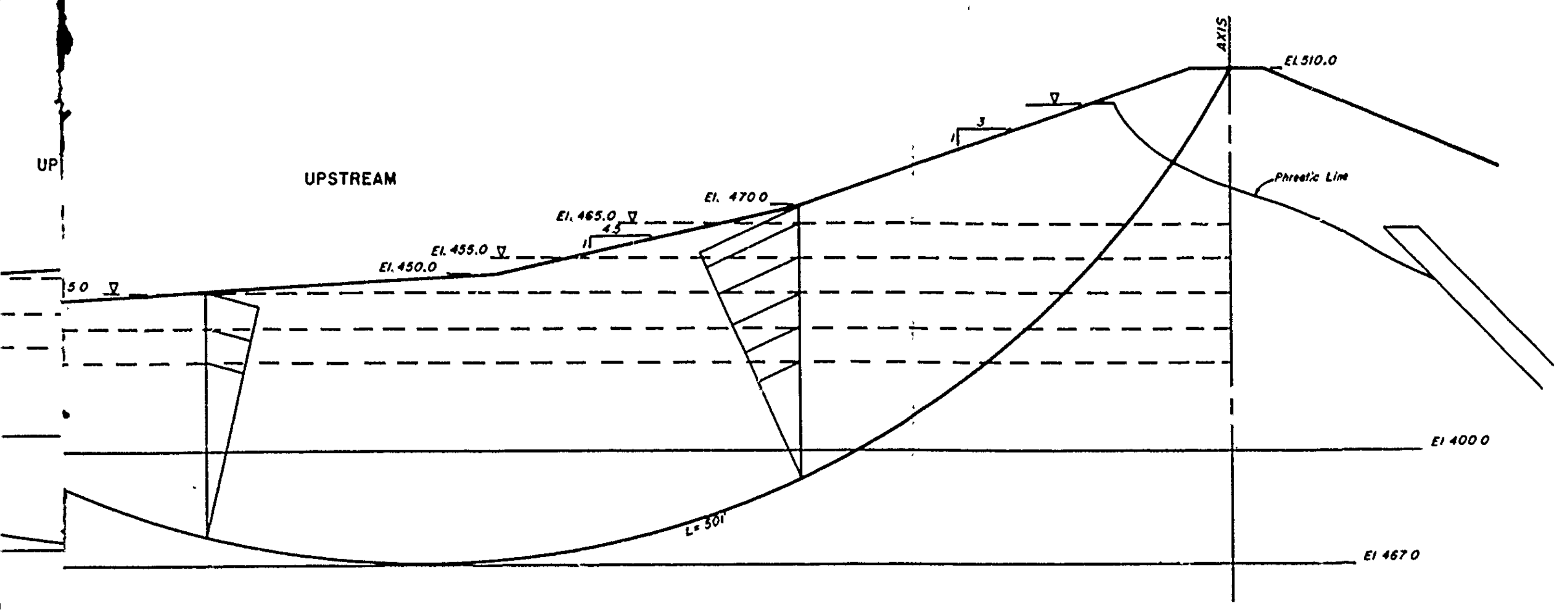
Segment	Area Ft²	Unit Wt PCF	Force Kips	Tan ϕ	N Tan. ϕ Kips	L Ft	C KSF	C Kips	N Tan ϕ + C Kips
Normal Forces									
A	5070	130	659						
B	19,390	676	1310						
			1969	0213	419	501	08	401	820
						ΣN Tan ϕ + C		=	820
Tangential Forces									
I	3350	130	435.5						
II	2800	676	189.5						
III	-2610	676	-176.5						
			ΣT = 448.5						

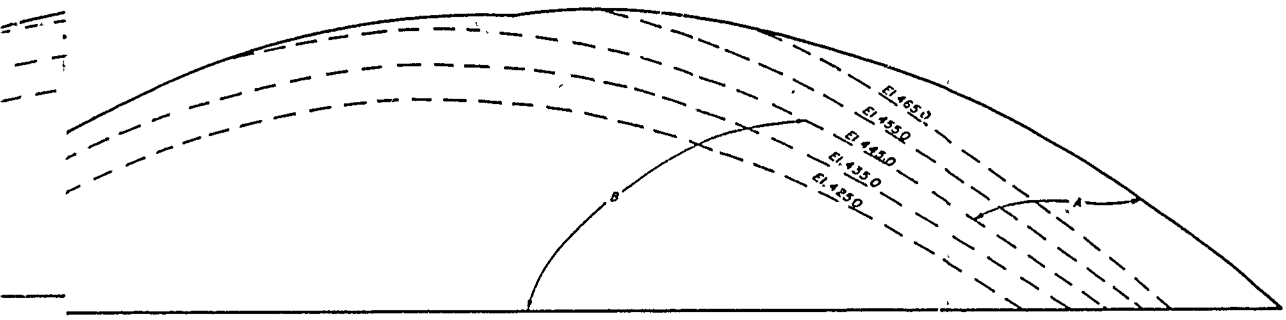
$$SF = \frac{\Sigma N \tan\phi + C}{\Sigma T} = \frac{820.0}{448.5} = 1.83$$



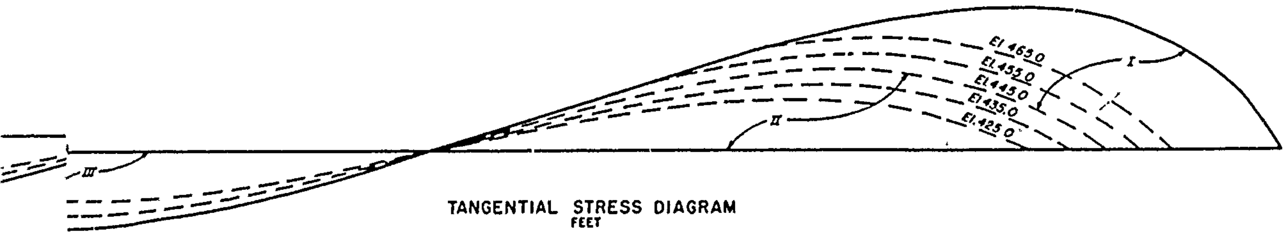




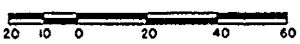




NORMAL STRESS DIAGRAM
FEET



TANGENTIAL STRESS DIAGRAM
FEET



DESIGN DATA

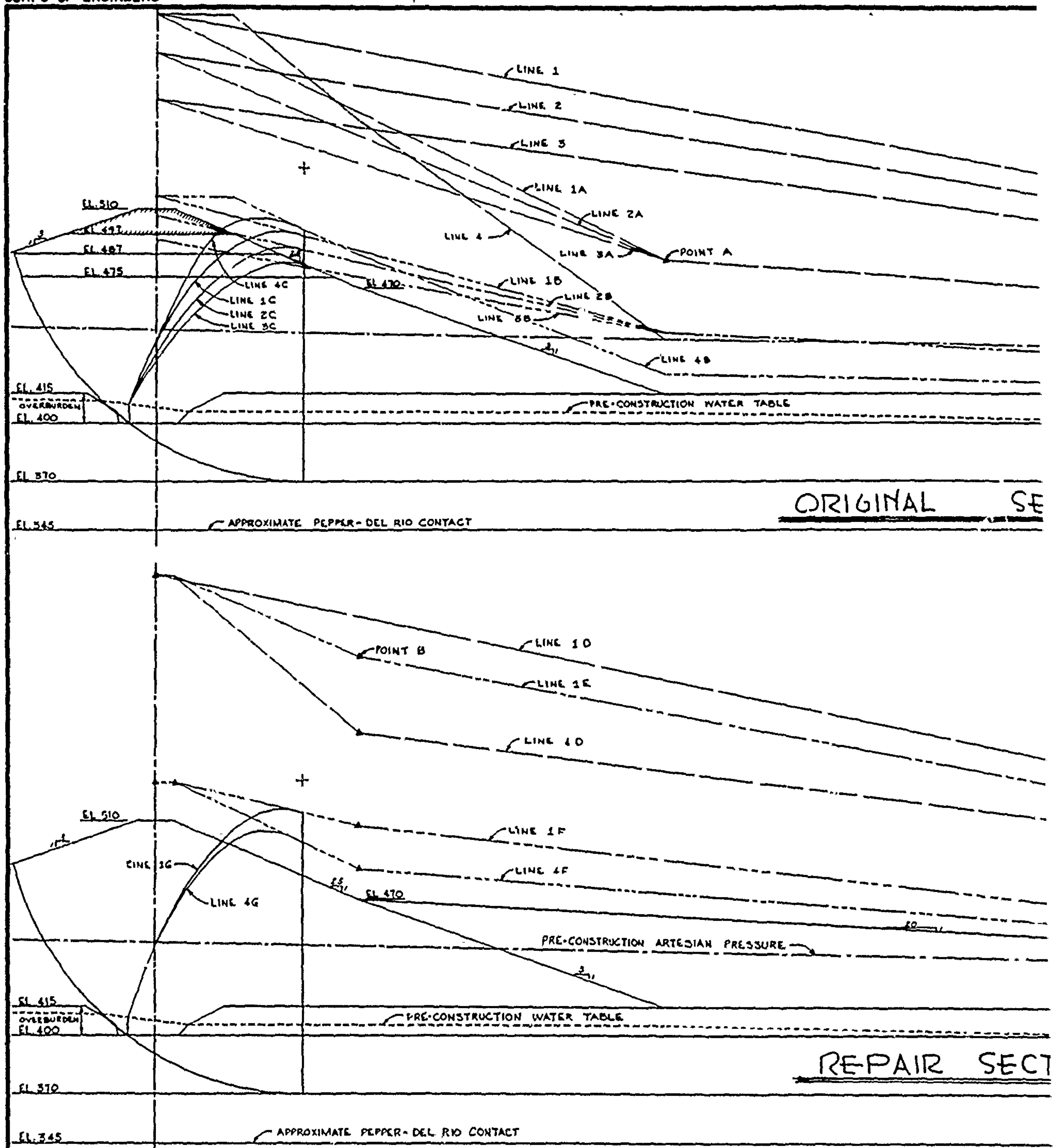
	"R" Strength	Unit Weight
Embankment	ϕ = 12.0° C = 0.4TSF	γ_m = 130.0 PCF γ_s = 130.0 PCF γ_b = 67.6 PCF

BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

STABILITY ANALYSIS
RIVER SECTION-UPSTREAM
CRITICAL POOL CONDITION
CIRCULAR ARC METHOD
SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH — JAN. 1963

CORPS OF ENGINEERS



PORE PRESSURE CONTOURS

LINE 1	The pore pressure at the Pepper-Del Rio contact based on 200 per cent of the height of fill (top of fill - El. 497) at the center line, added to the pre-construction artesian pressure and a straight line gradient assumed to P-64.
LINE 1-A	Line 1 was reduced at point A to limit the pore pressure to 200 percent of the height of the overlying material.
LINE 1-B	The pore pressure at El. 370 in the Pepper shale based on a straight line gradient from the pore pressure at the Pepper-Del Rio contact, (line 1-A) to the top of shale.
LINE 1-C	The pore pressure on the failure arc, assuming a straight line gradient from pore pressure at El. 370 (line 1-B) to top of shale.
LINE 1-D	The pore pressure of the Pepper-Del Rio contact based on 200 percent of the height of fill (Top of fill-El. 510) at the centerline, added to the pre-construction artesian pressure and a straight line gradient assumed to P-64.
LINE 1-E	Line 1-D was reduced at point B, and limited to 200 percent of height of overlying material including fill.
LINE 1-F	The pore pressure at El. 370 in the Pepper Shale base on a straight line gradient from the pore pressure at the Pepper-Del Rio contact, (LINE 1-E), to the top of shale.
LINE 1-G	The pore pressure on the arc, assuming a straight line gradient from the pore pressure at El. 370 (LINE 1-F) to the top of shale.
LINE 2	Comparable to line 1 series except based on top
LINE 3	Comparable to line 1 series except based on top
LINE 4	The pore pressure at the Pepper-Del Rio contact percent of the height of fill (Top of fill - El. 497 added to the pre-construction artesian press
LINE 4-A	Not pertinent to this line series.
LINE 4-B	The pore pressure at El 370 in the Pepper a straight line gradient from pore pressu Del Rio contact (LINE 4) to the top of shal
LINE 4-C	The pore pressure on the failure arc, assum line gradient from pore pressure at El. 3 to top of shale.
LINE 4-D	The pore pressure at the Pepper-Del Rio c on 200 percent of the height of fill (Top at all points added to the pre-constructio pressure.
LINE 4-E	Not pertinent to this line series.
LINE 4-F	The pore pressure at El. 370 in the Pepper a straight line gradient from the pore pr Pepper-Del Rio contact, (LINE 4-D), to the top
LINE 4-G	The pore pressure on the failure arc, assum line gradient from the pore pressure at El. to the top of shale.

U.S. ARMY

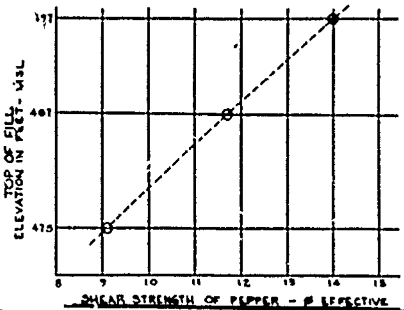
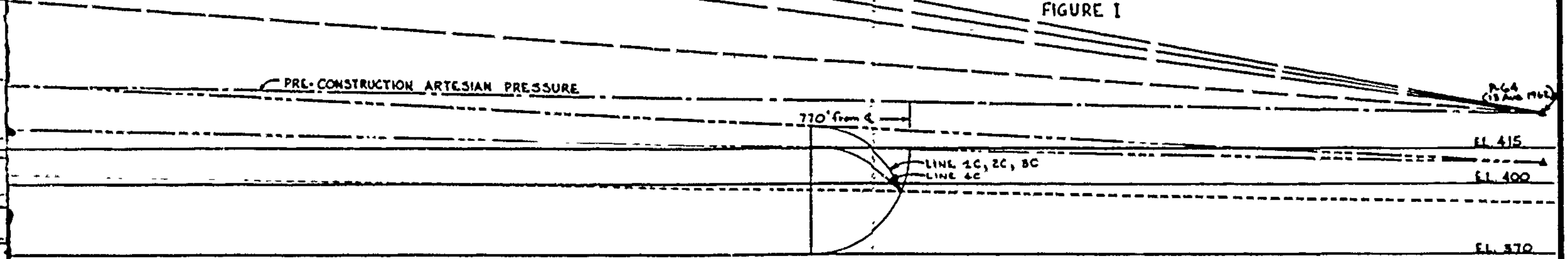
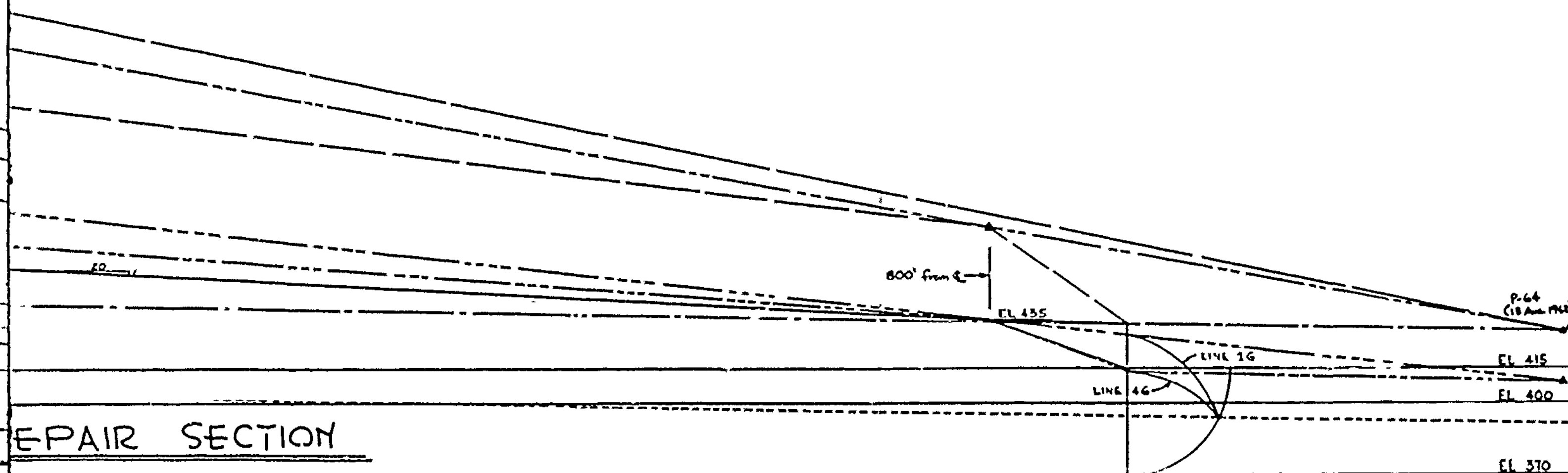


FIGURE I



IGINAL SECTION

EC



EPAIR SECTION

TIC

line 1 series except based on top of fill El. 487.

line 1 series except based on top of fill El. 475.

top of ... re at the Pepper-Del Rio contact based on 100 ... height of fill (Top of fill El. 497) at all points ... pre-construction artesian pressure.

top of ... to this line series.

act ba... 97) a... ssure at El 370 in the Pepper shale based ... essure. ...ne gradient from pore pressure at the Pepper- ...ct (LINE 4) to the top of shale.

r shal... sure ... ale. ...sure on the failure arc, assuming a straight ... from pore pressure at El. 370 (LINE 4-B) ...hale.

uming ... 370 ... ssure at the Pepper-Del Rio contact based ...ent of the height of fill (Top of Fill-El. 510) ... added to the pre-construction artesian

cont... op of F... ion a... to this line series.

...ssure at El. 370 in the Pepper shale based on ...ne gradient from the pore pressure at the ...o contact, (LINE 4-D), to the top of shale.

per sh... pressu... op of ... ssure on the failure arc, assuming a straight ... from the pore pressure at El. 370 (LINE 4-F) ... shale.

m n a... El. 370

RESULTS

1. The slide was analyzed by a simplified procedure for a circular arc plane failure surface to determine the shear strength required for stability (SF=1.0). The shear strength thus determined was applied to the repair section using comparable pore pressure assumptions to determine the safety factor.

2. The slide was analyzed for three elevations for the top of fill to show variation of strength required as construction of fill progressed. The results are summarized in Figure 1. The pore pressures used are indentified as line series 1, 2, and 3. The repair section was analyzed using only the shear strength determined from the analysis of the slide with the fill at El. 497, and a saftey factor = 1.72 was obtained.

3. The slide was analyzed using pore pressure line series 4, and the shear strength in the Pepper shale was determined to be ϕ (effective) = 9.5 degrees. This value was applied to the repair section and a safety factor = 1.37 was obtained.

DESIGN DATA

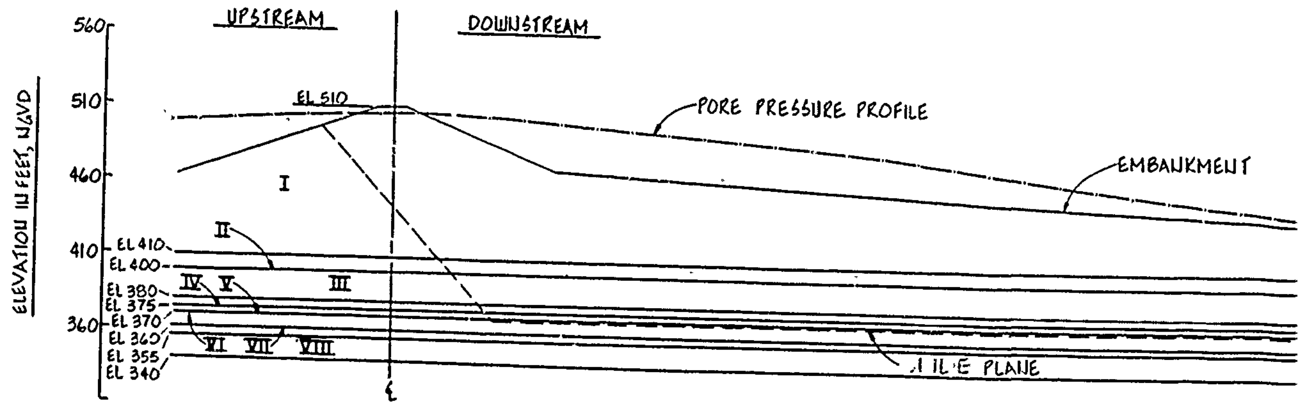
UNIT WEIGHT SOIL = 130 pcf
UNIT WEIGHT WATER = 62.5 pcf
SHEAR STRENGTH :
EMBANKMENT : Pre-slide, c = 0.9 tsf = 1.8 ksf
EMBANKMENT . New construction above El 450 berms and overburden c = 1.0 tsf = 2.0 ksf
EMBANKMENT : In failure zone below El. 450 c = 0.4 tsf = 0.8 ksf
PEPPER SHALE : As shown.

BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS

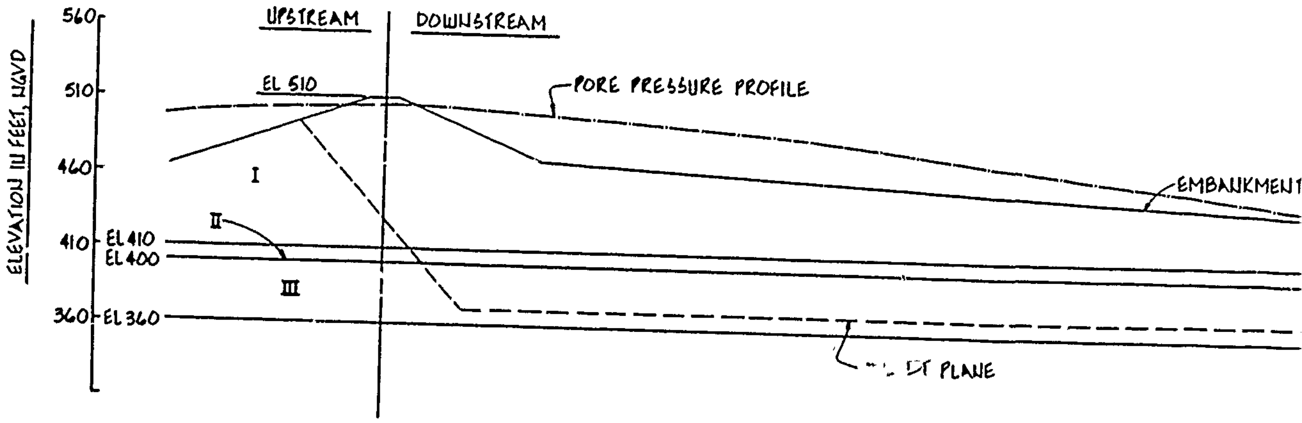
STABILITY ANALYSES

EMBANKMENT SLIDE AREA - STATION 55+00

U.S. ARMY ENGINEER DISTRICT, FORT WORTH

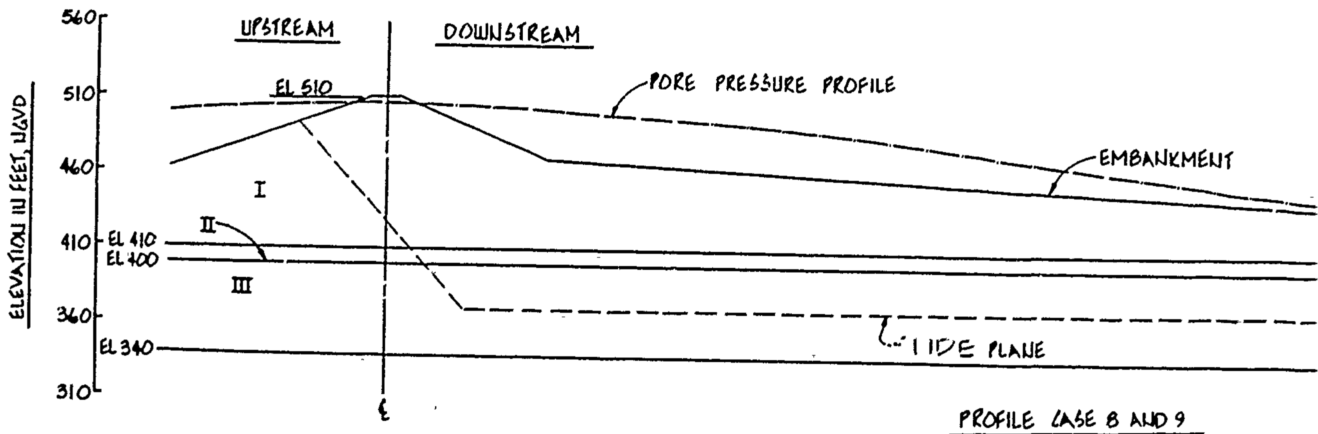


PROFILE CASE 1 THRU 5

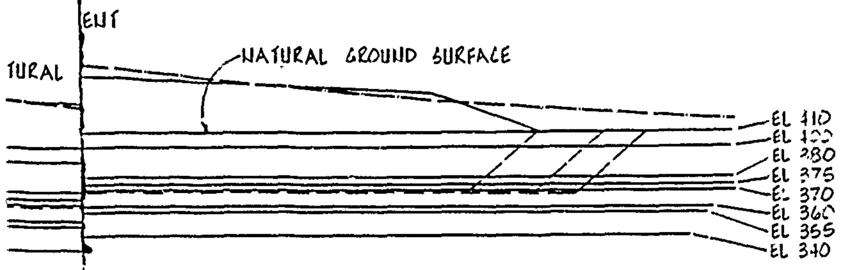


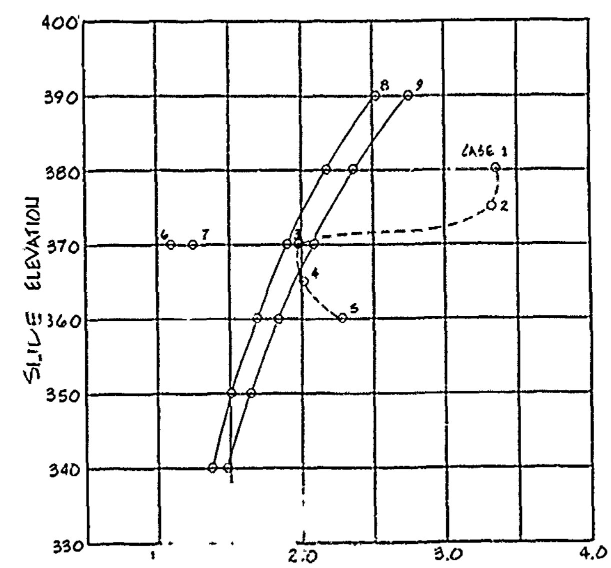
RESIDUAL STRENGTH ASSUMED

PROFILE CASE 6 AND 7



PROFILE CASE 8 AND 9

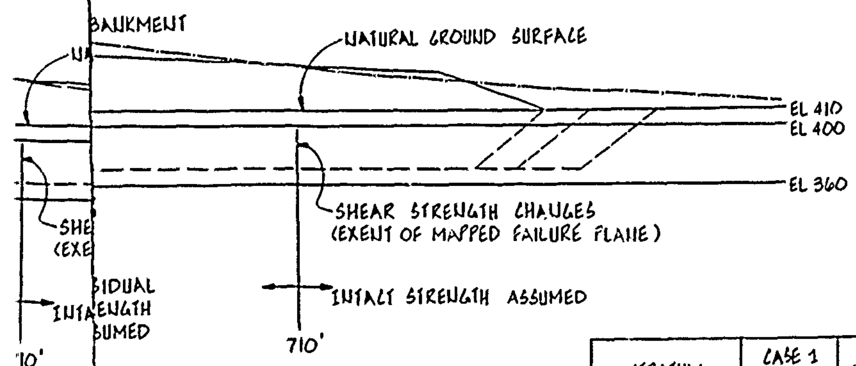




NOTES:

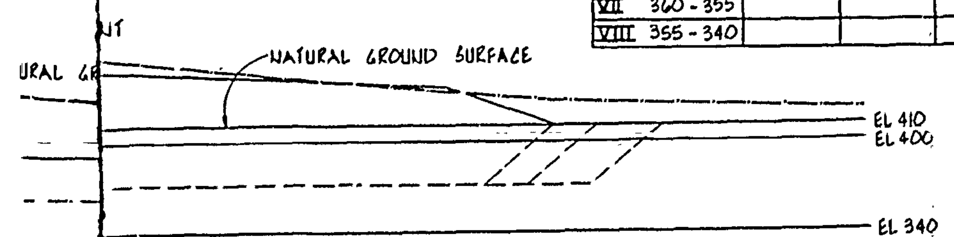
1. FOR GENERAL NOTES SEE PLATE

*2. THE STRENGTH CHANGES AT 710 FEET DOWNSTREAM. THE STRENGTH CHANGES FROM LOW TO HIGH SIGNIFYING THE CHANGE FROM RESIDUAL TO INTACT STRENGTH.



SHEAR STRENGTH SUMMARY

STRATUM	CASE 1 φ° C,PSF		CASE 2 φ° C,PSF		CASE 3 φ° C,PSF		CASE 4 φ° C,PSF		CASE 5 φ° C,PSF		CASE 6 φ° C,PSF		CASE 7 φ° C,PSF		CASE 8 φ° C,PSF		CASE 9 φ° C,PSF	
I EMBANKMENT	3	2000	3	2000	3	2000	3	2000	3	2000	3	1000	3	2000	3	2000	3	2000
II 410 - 400	20	400	20	400	20	400	20	400	20	400	20	400	20	400	20	400	20	400
III 400 - 380	18	580	18	580	18	580	18	580	18	580	8 14	0* 400	8 11	0* 400	14	400	16	400
IV 380 - 375	23	760	23	760	23	760	23	760	23	760	8 14	0* 400	8 14	0* 400	14	400	16	400
V 375 - 370			23	1000	18	600	18	600	18	600	8 14	0* 400	8 14	0* 400	14	400	16	400
VI 370 - 360					14	400	15	480	15	500	8 14	0* 400	8 14	0* 400	14	400	16	400
VII 360 - 355							15	500	21	620					14	400	16	400
VIII 355 - 340									22	640					14	400	16	400



50 0 50 100

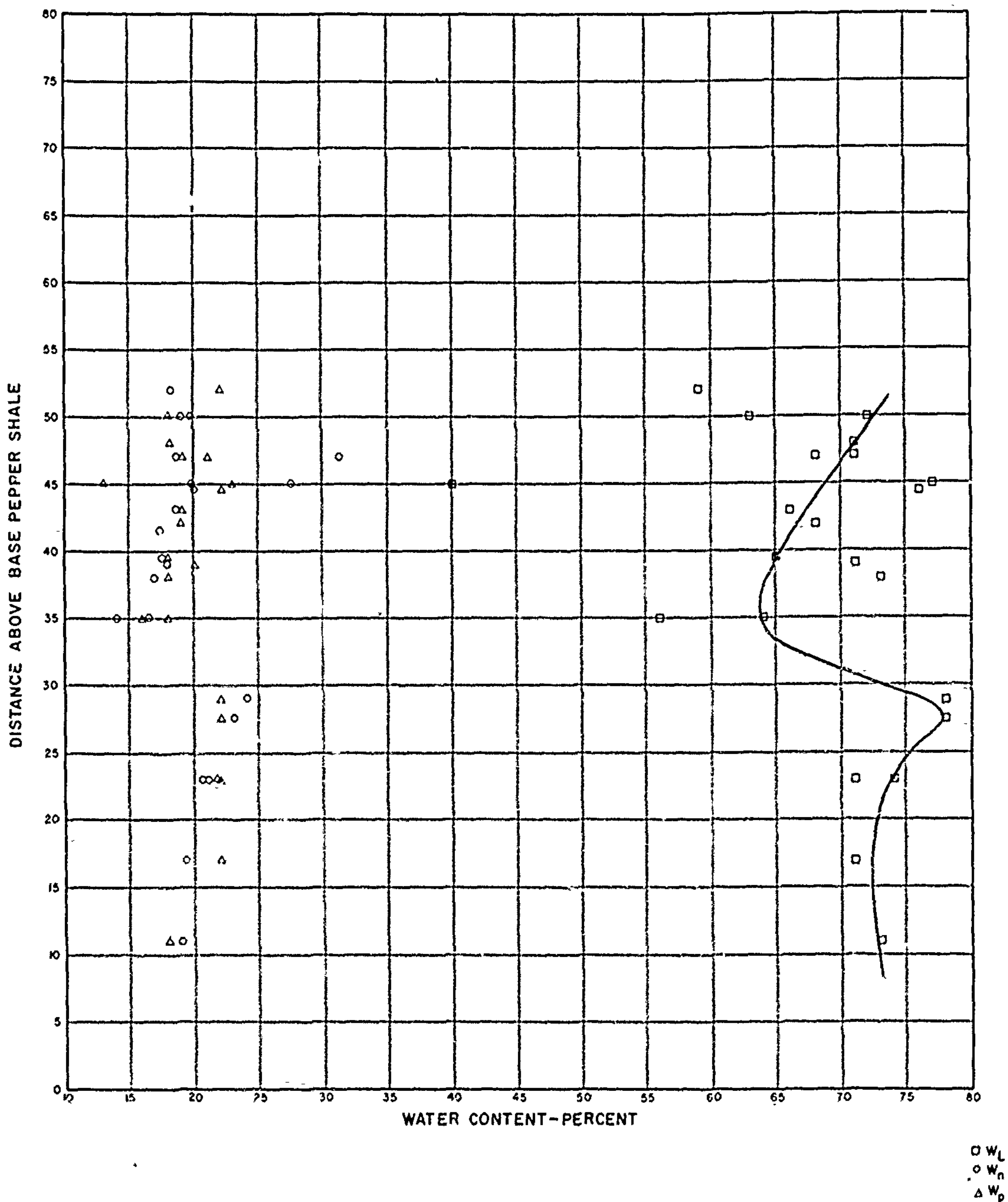
SCALE IN FEET

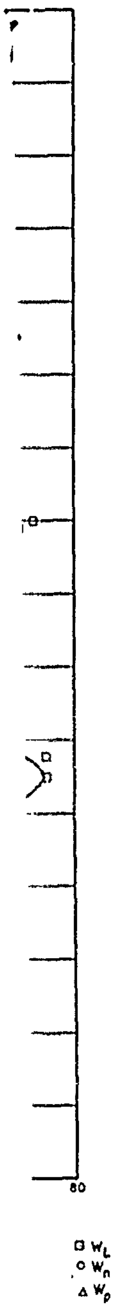
U.S. ARMY ENGINEER DISTRICT, FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

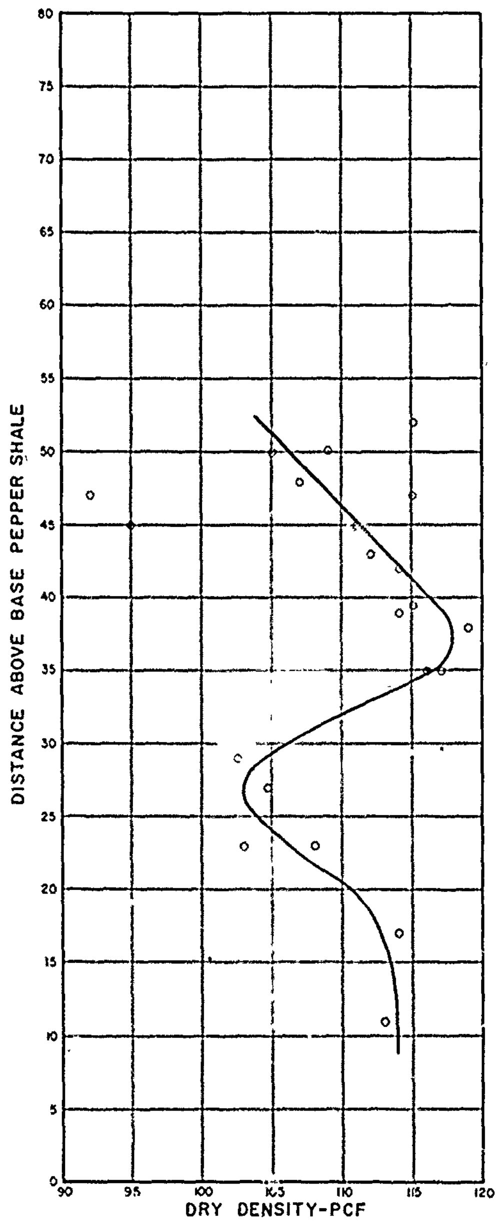
WACO DAM
BOSQUE RIVER, TEXAS

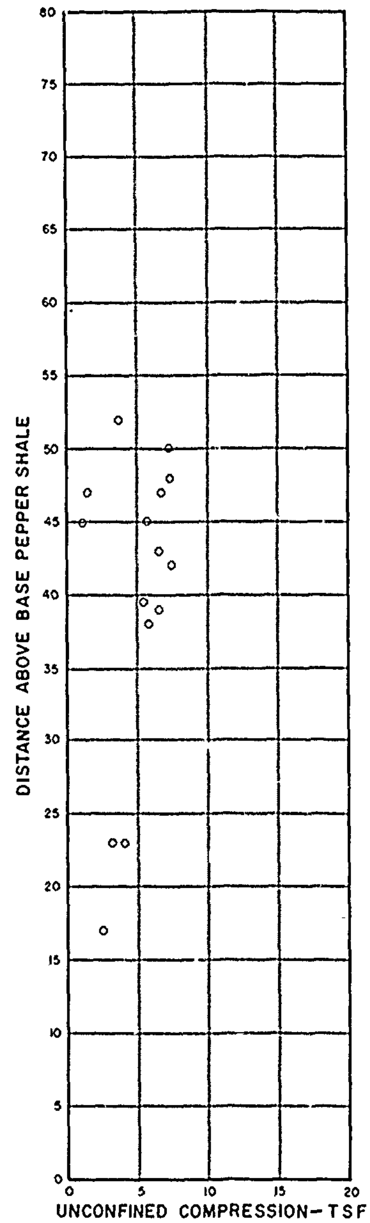
STABILITY ANALYSES
1984 CONDITION

DESIGNED BY:
DRAWN BY:
REVIEWED BY:
SUBMITTED BY:
ENGINEER:
INV. NO.
CONTR. NO.
DRAWING NUMBER
DATED:
SHEET NO.


DISTANCE ABOVE BASE PEPPER SHALE
WATER CONTENT-PERCENT
W_L
W_n
W_p







BRAZOS RIVER AND TRIBUTARIES, TEXAS

WACO DAM

BOSQUE RIVER, TEXAS

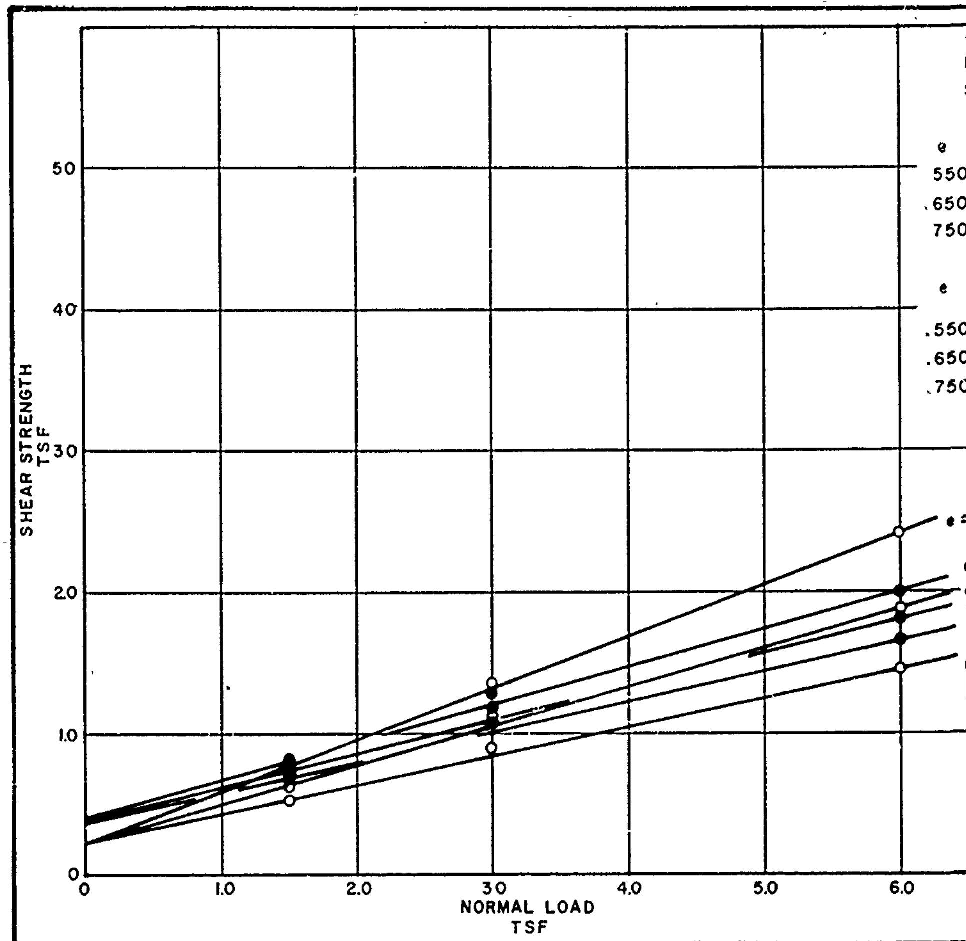
PHYSICAL CHARACTERISTICS

PEPPER SHALE

BETWEEN FAULTS WITHIN SLIDE AREA

SCALES AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT WORTH JAN. 1963



Time ≅ 180 min.
Rate ≅ 0.0003
Specimen Size
○ = 0.25'

e	c	tan Ø	Ø
550	0.22	.360	19.8°
.650	0.22	.276	15.4°
750	0.22	.206	11.6°

● = 0.50

e	c	tan Ø	Ø
.550	.39	.267	14.9°
.650	.39	.240	13.5°
.750	.39	.211	11.9°

BRAZOS RIVER AND TRIBUTARIE
WACO DAM
BOSQUE RIVER, TEXAS
PEPPER SHALE
BETWEEN NORTH & SOUTH
SHEAR STRENGTH VS. NORM
"S" TESTS
SCALE AS SHOWN

U.S. ARMY ENGINEER DISTRICT, FORT

Time ≅ 180 min.
Rate ≅ 0.0003
Specimen Size
○ = 0.25'

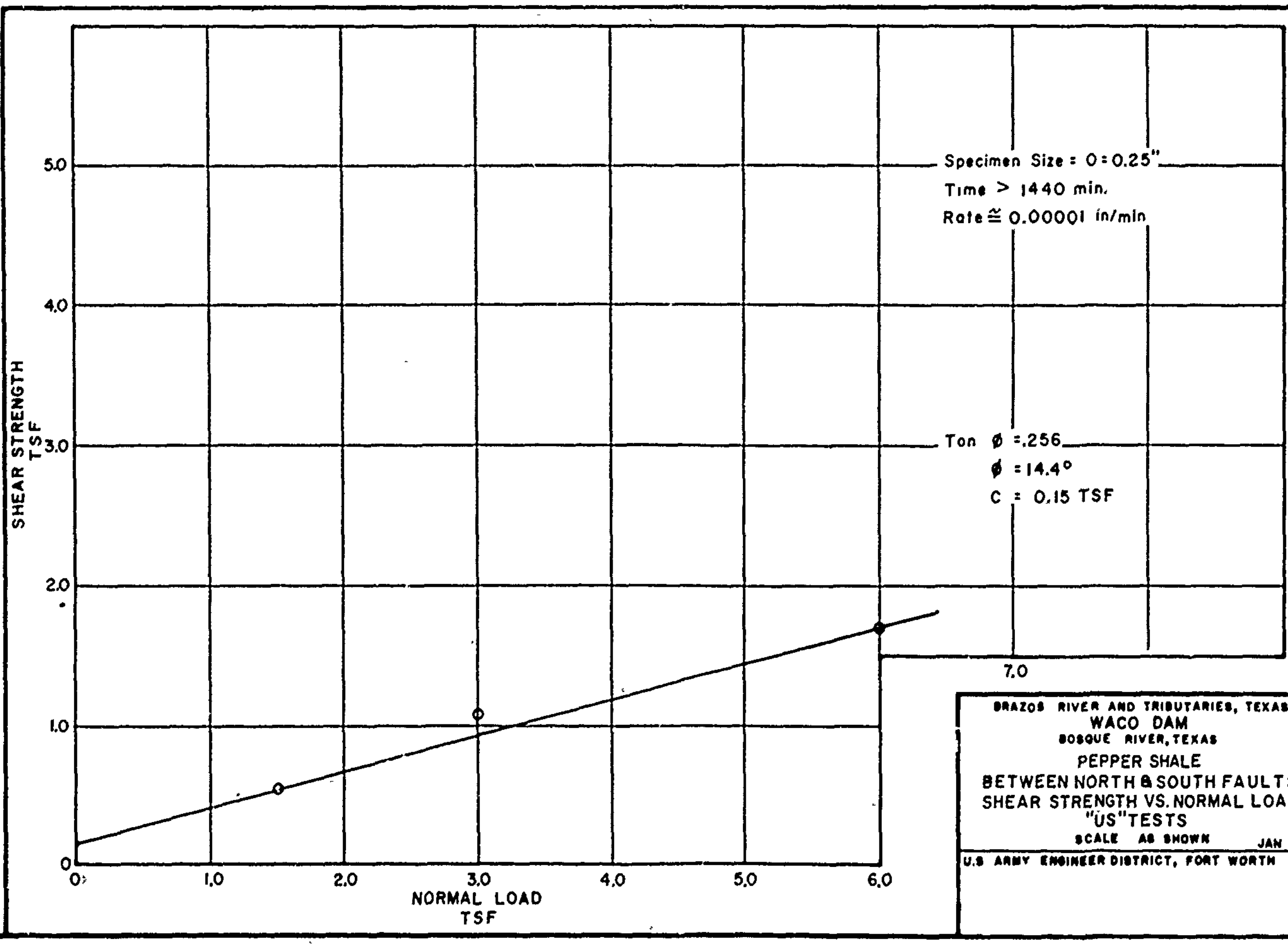
c	tan Ø	Ø
0.22	.360	19.8°
0.22	.276	15.4°
0.22	.206	11.6°

● = 0.50

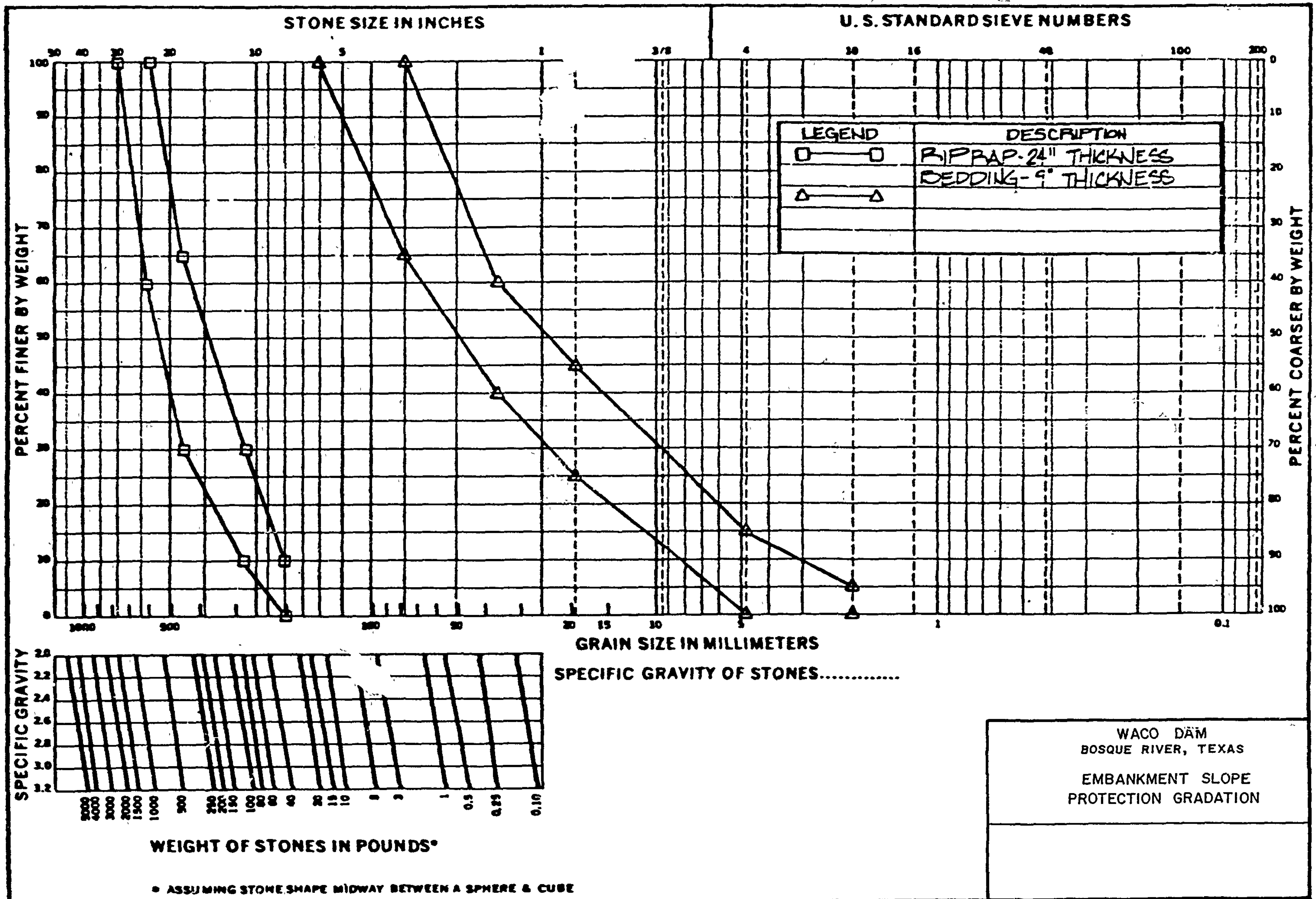
c	tan Ø	Ø
.39	.267	14.9°
.39	.240	13.5°
39	.211	11.9°

.550
= 550
=.650
=.650
=.750
=.750

BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS
PEPPER SHALE
BETWEEN NORTH & SOUTH FAULTS
SHEAR STRENGTH VS. NORMAL LOAD
"S" TESTS
SCALE AS SHOWN JAN 1963
U.S. ARMY ENGINEER DISTRICT, FORT WORTH



BRAZOS RIVER AND TRIBUTARIES, TEXAS
WACO DAM
BOSQUE RIVER, TEXAS
PEPPER SHALE
BETWEEN NORTH & SOUTH FAULTS
SHEAR STRENGTH VS. NORMAL LOAD
"US" TESTS
SCALE AS SHOWN JAN 1963
U.S. ARMY ENGINEER DISTRICT, FORT WORTH



WACO DAM
BOSQUE RIVER, TEXAS

EMBANKMENT SLOPE
PROTECTION GRADATION

ENG FORM 4056
APR 67

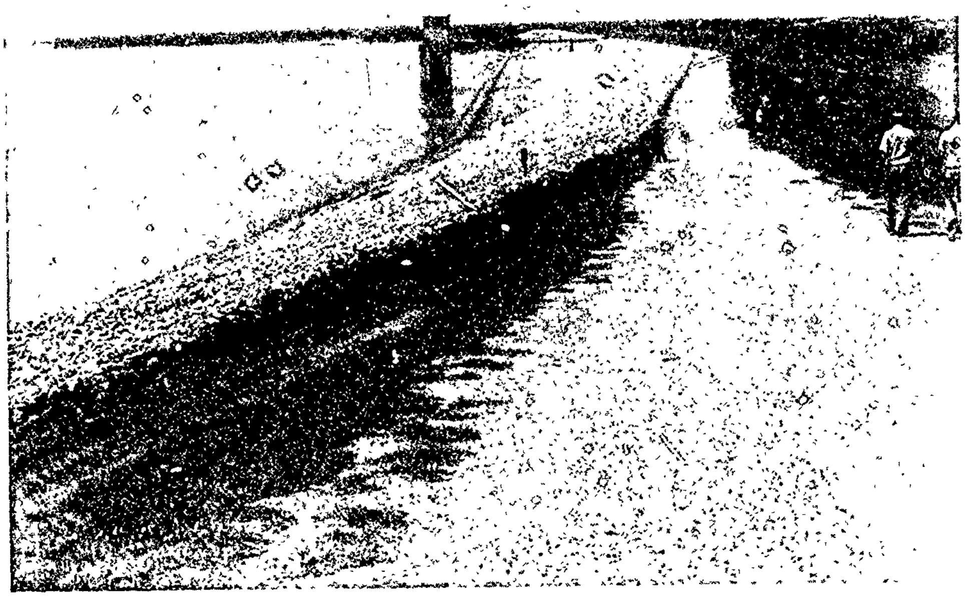


PHOTO 1. Looking at the U/S ripraped slope from the right abutment with outlet works tower in background, Sep. 83

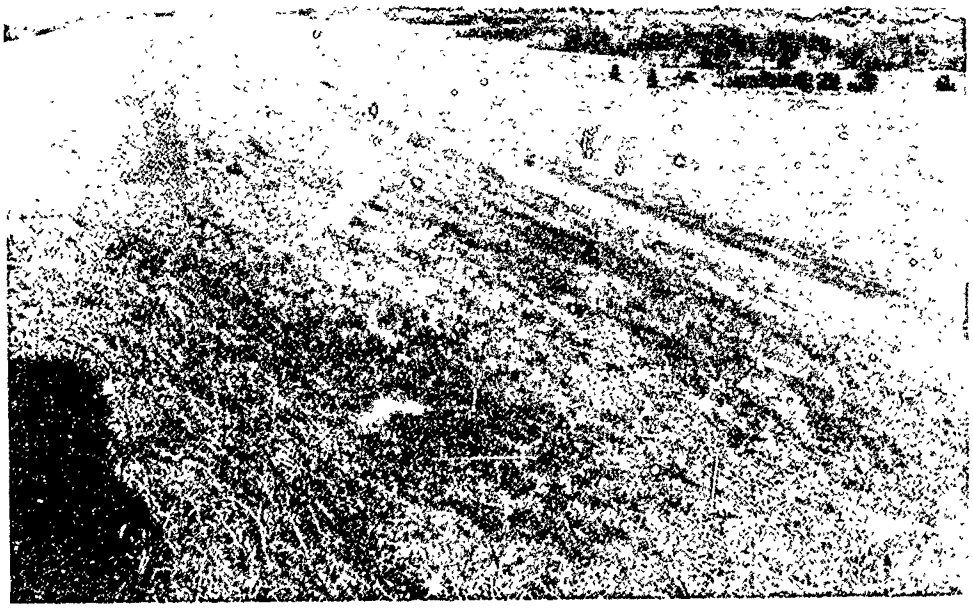


PHOTO 2. View of the grass covered D/S slopes of dam. Note the flat berm in the background over the old slide area, Sep 83

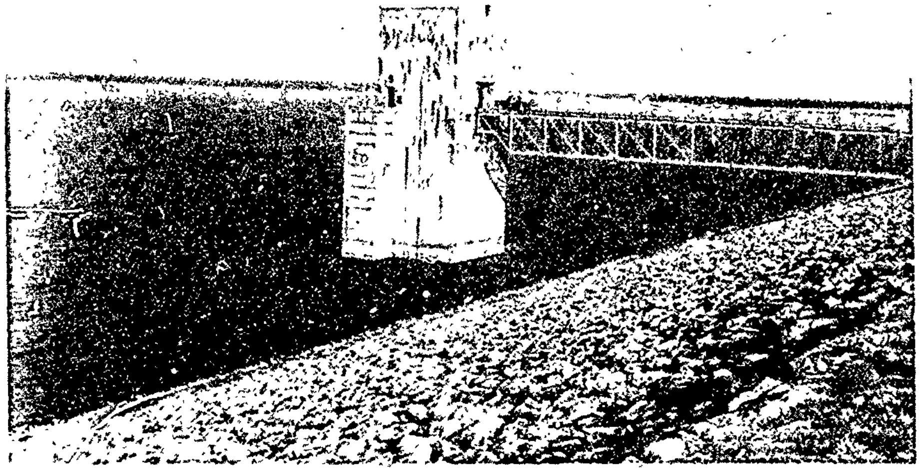


PHOTO 3. View of the outlet works tower and service bridge, Sep 83.

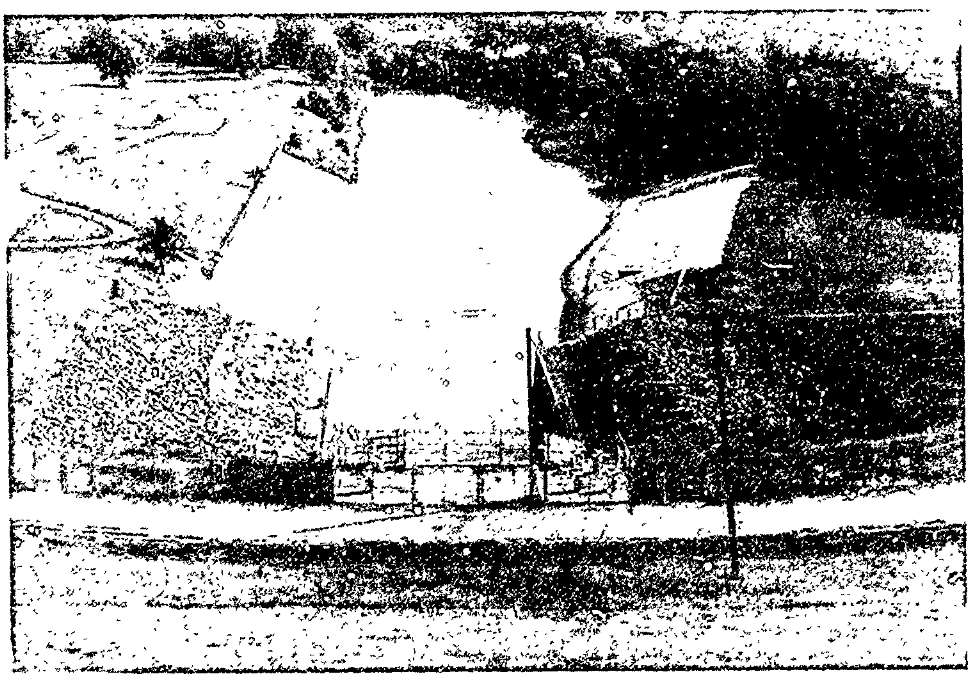


PHOTO 4. View of the outlet works discharge channel, Sep 83.

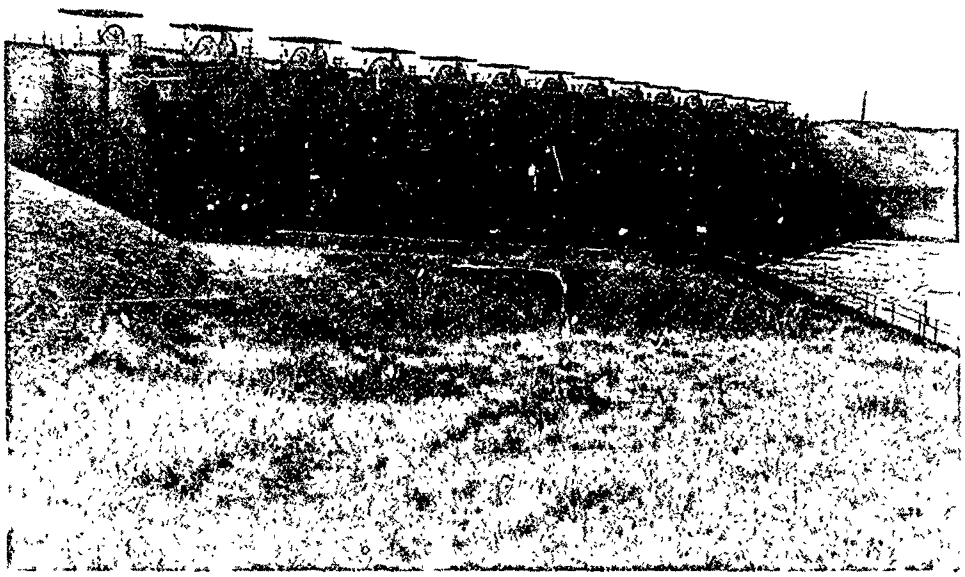


PHOTO 5. Downstream View of the gated spillway structure Sep 87.

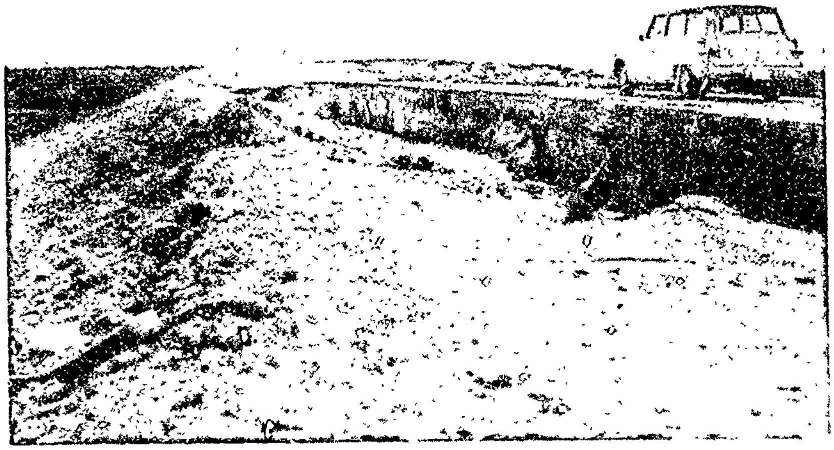


PHOTO 6. Crest of dam, slide area about sta 55+00, Oct 61.

PHOTO 7. Upstream crest of dam at sta 59+00, dropoff in the crack is about 19' deep, Oct 61.

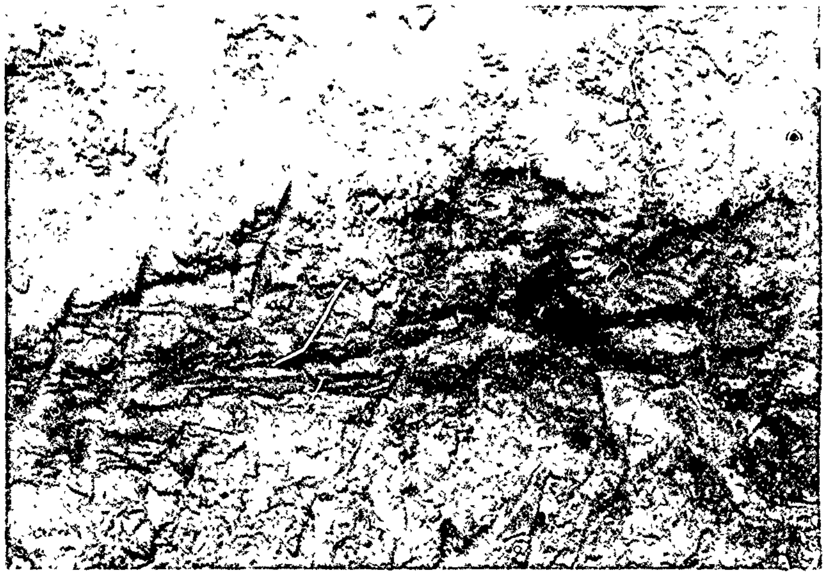


PHOTO 8. Pepper shale/Del Rio shale contact in inspection hole at sta 58+60.

PHOTO 9. South Fault in South Fault trench excavation, downstream. Pepper shale on left against Eagle Ford shale on right.

PHOTO 11. Upturned Eagle Ford shale on the down-thrown side of the South Fault in the upstream fault excavation.

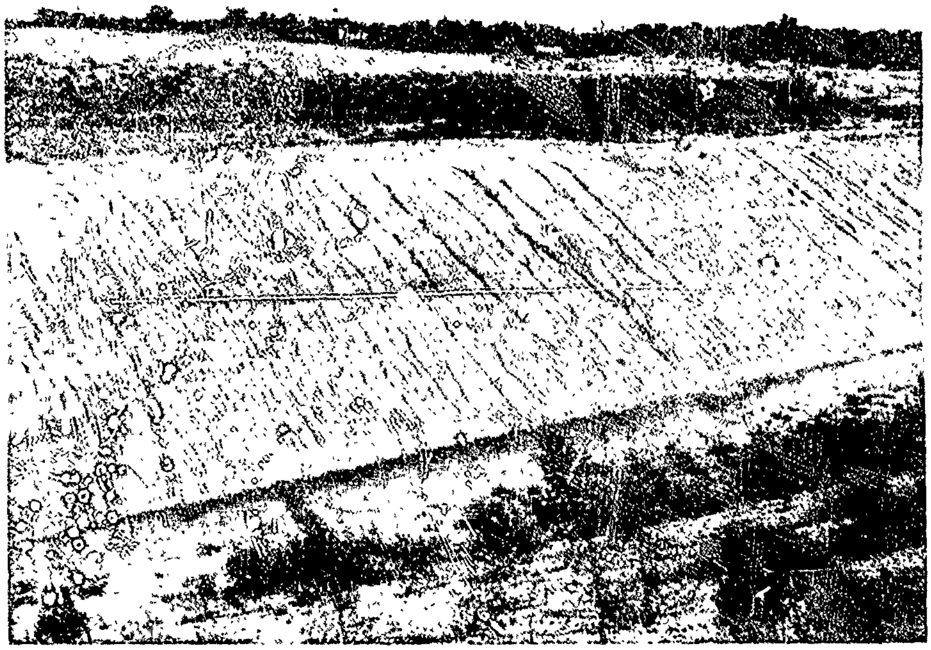


PHOTO 13. Oxidized sandstone seam in the Pepper shale in the middle inspection trench 1100 feet D/S from dam axis station 55+00.

PHOTO 14. Eagle Ford shale (left)/overburden contact in the embankment foundation; vicinity of station 20+00 on centerline.

PHOTO 15. Vertical joint with weathering halo in the Eagle Ford shale; embankment foundation about station 32+00, D/S.

PHOTO 16. Slickenside in the Pepper shale below and nearly normal to the failure plane in inspection hole sta 58+20.

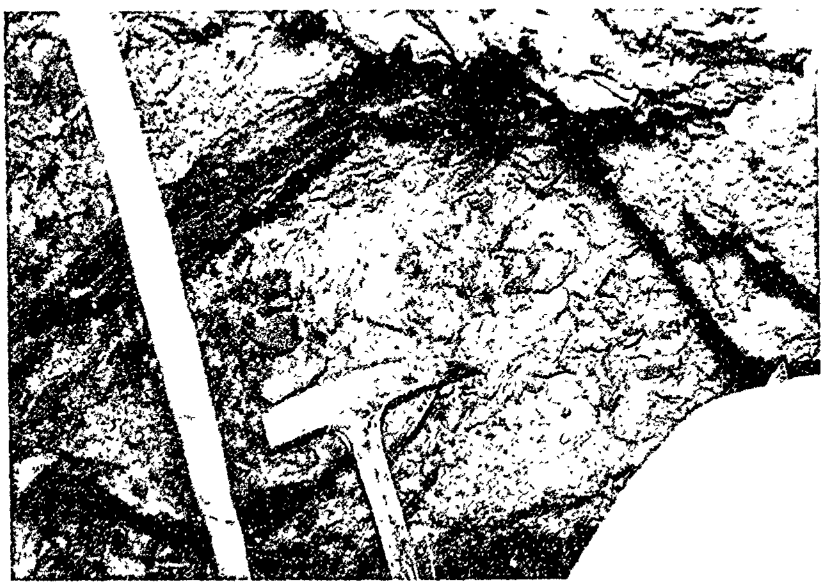


PHOTO 17. Slickenside 2-1/2 feet below and nearly paralleling the failure plane in inspection hole sta 58+20.

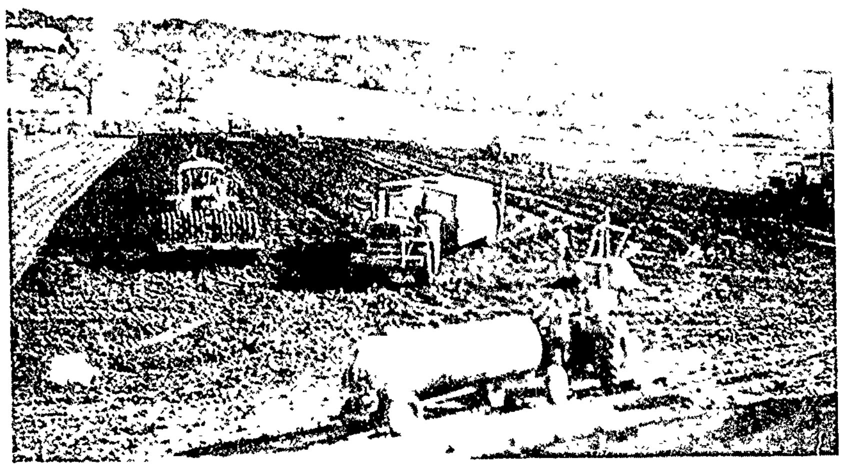


PHOTO 18. Fill placement in the area between the outlet works and river, Nov 63.

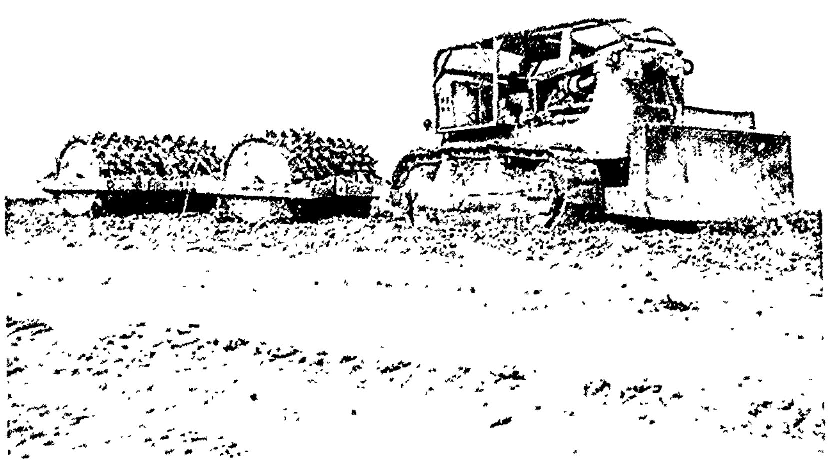


PHOTO 19. Fill placement w/twin sheepsfoot rollers, Jan 63.

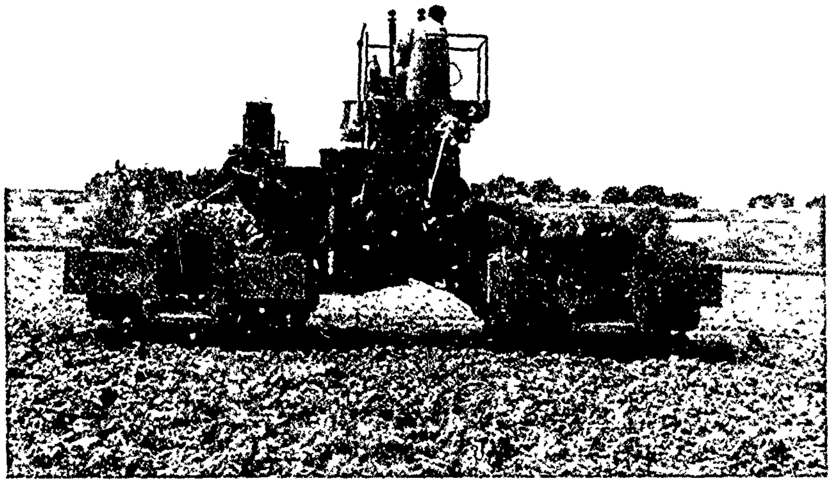


PHOTO 20. LeTorneau equipment M50-55
Power Packer electric drive drums,
compacting fill, Apr 63

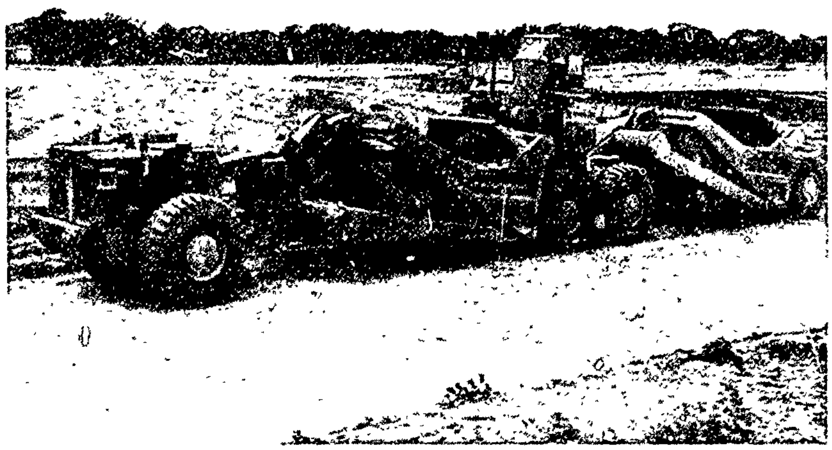


PHOTO 21. LeTorneau equipment L-140 electric digger,
Jun 63.

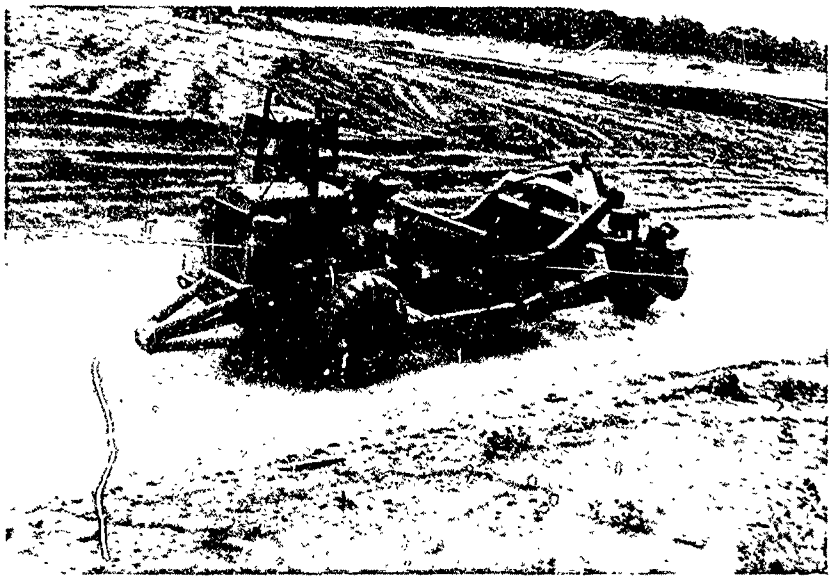


PHOTO 22. Scraper, R. G. Letorneau equipment LTU-27 electric digger 3 wheel drive, Apr 63

PHOTO 23. R. G. Le Torneau equipment, K-53F tractor 3 wheel drive pulling 50 ton Ferguson pneumatic roller, Apr 63.

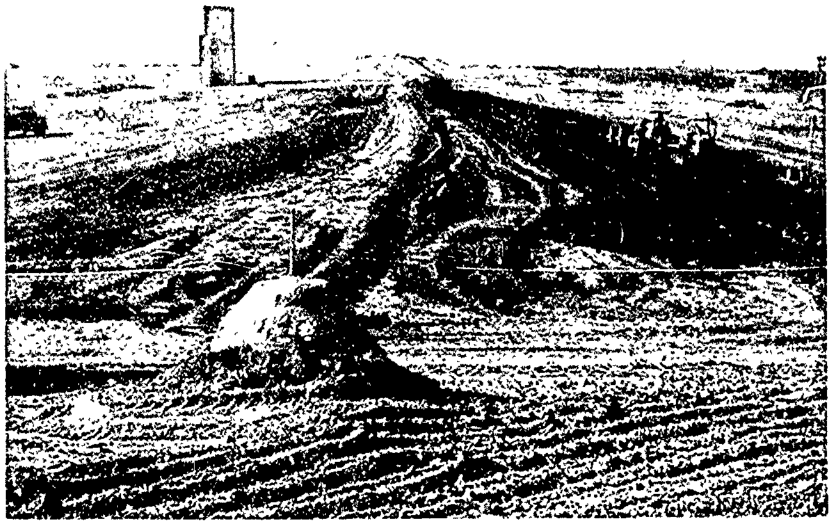


PHOTO 24. Fill placement in closure area about sta. 15+00, piezometer mound in foreground, Oct 64.

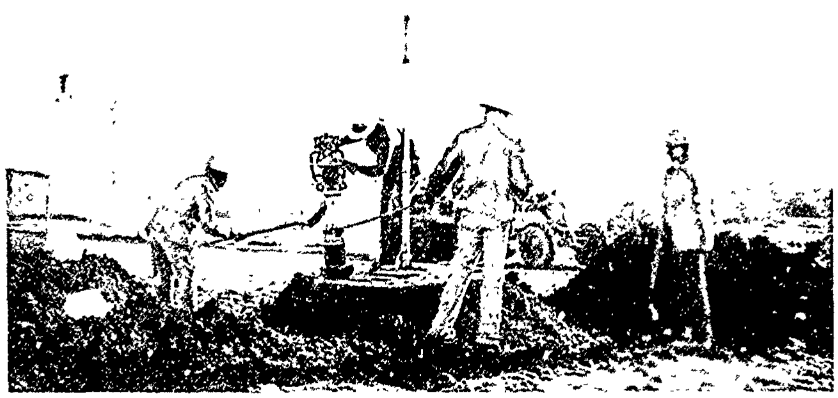


PHOTO 25. Fill placement of mound around piezometer, Jan 63

PHOTO 26. Austin chalk exposed on the right abutment, Feb 64

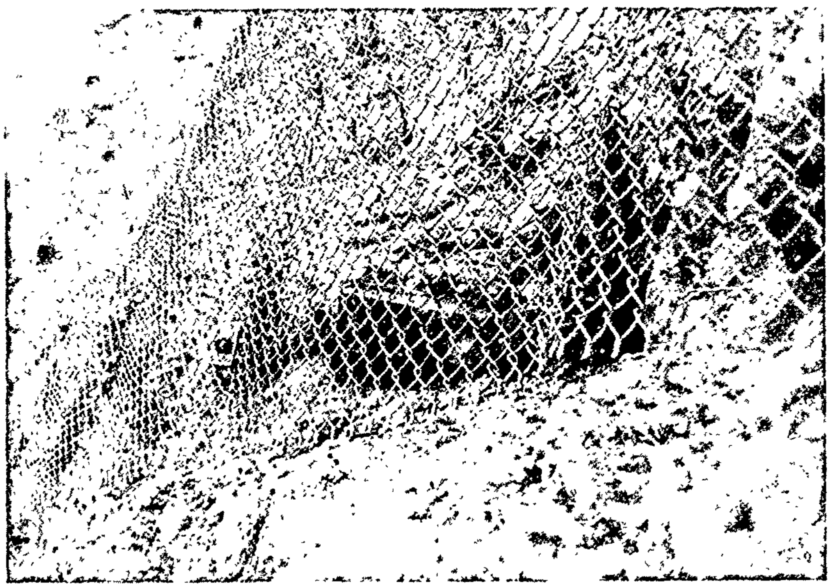


PHOTO 27. Rock bolts and protective wire mesh, Feb 64

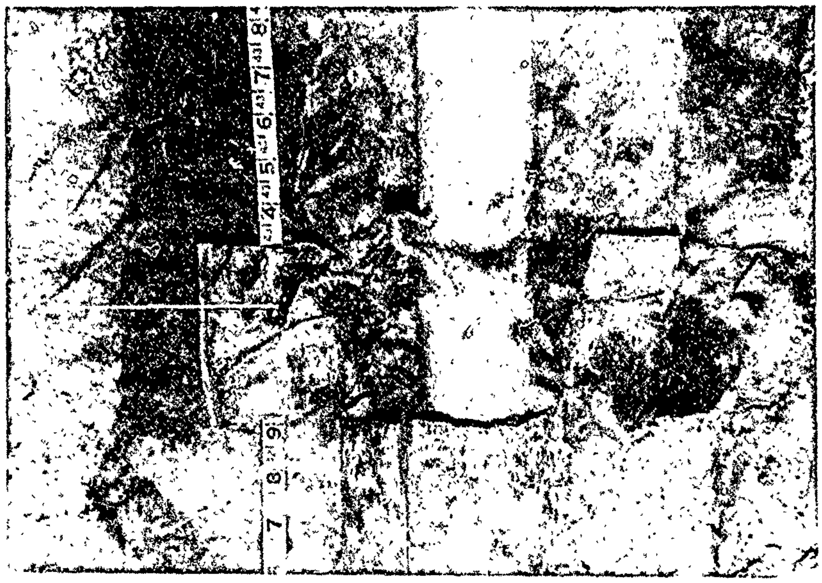


PHOTO 28. View of slope indicator SI-1 showing pinched pipe due to compression, Aug 63.

PHOTO 29. View of slope indic a 55+00, collapse at fill/ov ntact, Aug 63.